Proceedings

Chicago Committee on High Rise Buildiings

Seminar on Recladding of the Amoco Building in Chicago, Illinois

15

November 1995
Report No. 15

FORWARD

The Chicago Committee on High Rise Buildings recently celebrated the 25th Anniversary of its founding. The committee is proud of the work it has accomplished and is looking forward to the ever increasing challenges of the next quarter century. The Committee has also re-dedicated itself to its founding philosophy ---- "to initiate and support research for the economic design, construction and maintenance of high rise structures."

The "High Rise Committee" was founded by 25 outstanding professionals in the Chicago building industry, to seek improved and more economical approaches to all aspects of high rise construction - from the inception of the design to the project's completion and maintenance.

The background of the Committee members insures a full range of expertise from every facet of the building industry. Architect, engineers (structural, mechanical and electrical), government officials, developers, contractors, material suppliers and other industry representatives ---- all volunteer their time and professional knowledge to this concerted drive to better the industry.

Specific problem areas of high rise development have been investigated, including structural system safety factors, mechanical and electrical systems, wind effects on high rise buildings, exterior claddings, life safety, and numerous other aspects of design and maintenance. The Committee will continue to investigate these types of problems and disseminate the resulting information in publications to interested groups in the building industry.

The High Rise Committee also serves as a central organization to coordinate the sharing, correlation and evaluation of research data from all sources, and to stimulate continued cooperation and innovation by industry, educational institutions, research organizations and building professionals.

While the Chicago Committee on High Rise Buildings directs its efforts specifically toward the Chicago building industry and construction environment, our findings are available to interested parties throughout the country.

A non-profit organization, the Committee welcomes professional assistance and industry support. Through such cooperation, it strives to expand the knowledge of all aspects of the building industry, and to achieve the research and innovation that will bring about improved products, techniques and greater economy in high rise development.

Chicago Committee on High Rise Buildings

Gerald L. Johnson, Chairman

PUBLICATIONS

Following are other publications by The Chicago Committee on High Rise Buildings available through the Committee Secretary:

REPORT #3
Strain Measurements in Rock-Soaked Caissons
Historic Document(1972 superseded by #10

REPORT #6
Seminar on Chicago High Rise Code:
Historic Document (1977)

REPORT #7
Energy Use and Management in High Rise Buildings - Part I: New Construction
Historic Document (1981)

REPORT #8
Energy Use and Management in High Rise Buildings - Part II: Retrofit
Historic Document (1982)

REPORT #9
History of Chicago Building Foundations
(1984)

Developments in foundation design and construction for high rise building in Chicago are described with particular emphasis on the post war period of 1948-1983. The improvements in on-site testing techniques and collation of predictious and preferences that have led to significantly higher foundation design bearing pressures are documented. Soil and foundation related construction problems influencing design and construction procedures are illustrated with case histories.

REPORT #10
Full Scale Load Test on Caisson Foundation
(1986)

An instrumented full scale load test on a belled caisson on hardpan is described. The maximized test load of 1270 tons, 30 inch shaft and 6 foot diameter bell is believed to be the heaviest known load test on soil. Strain gauges and pressure cells were used to monitor stresses and strains in the caisson bell during loading in an effort to determine at what pressure tension stresses become critical.

REPORT #11
Fire Endurance on High Strength Concrete(1987)

With the increased use of high-strength concrete, concern has developed regarding performance of such when exposed to fire. More specifically, questions regarding the fire endurance of concrete containing silica fumes since a published report concluded that these concretes explode when subjected to elevated temperatures. This report presents the results of standard fire tests on concretes with and without silica fumes with normal compressive strengths of 9,000 and 1,4000 psi.

REPORT #12
Exterior Cladding on High Rise Buildings
Proceedings - November,1989 Symposium
(1990)

REPORT #13
Composite Steel and Concrete Construction
Proceedings - October, 1992 Symposium
(1993)

REPORT #14
Indoor Environment Seminar
Proceedings - October, 1992 Symposium
(1993)

PROCEEDINGS OF THE SEMINAR
ON THE
RECLADDING OF THE AMOCO BUILDING IN CHICAGO, IL
HELD ON NOVEMBER 11, 1993

SPONSORED BY
THE CHICAGO COMMITTEE ON HIGH RISE BUILDINGS
WITH THE APPROVAL AND ACTIVE PARTICIPATION OF
AMOCO CORPORATION

EDITOR
IAN R. CHIN
EXTERIOR WALL TASK FORCE CHAIRMAN
THE CHICAGO COMMITTEE ON HIGH RISE BUILDINGS

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PREFACE

In December 1972, construction of the 80-story tall Amoco building in Chicago, IL was completed. Approximately 20 years later in September 1991, removal and replacement of all of the original 44,000 marble panels on the exterior facade on the building with new granite panels was completed.

On November 11, 1993, a seminar was held in Chicago, IL, to present information on the design of the marble panels; on the investigation performed to determine the causes of the failure of the marble panels; on why the marble panels had to be removed; and on the design, testing, and construction of the new granite panels used to replace the original marble panels.

The seminar was sponsored by the Chicago Committee on High Rise Buildings with the approval and active participation of Amoco, the Owner of the building.

The speakers at the seminar consisted of representatives of:

1. Amoco Corporation

   Owner of Building

2. Wiss, Janney, Elstner Associates, Inc.

   Architect and structural engineer that investigated the original marble panels and designed the new granite panels.

3. Schal Bovis, Inc.

   Construction Manager of Recladding

4. W. R. Weis Company

   Marble Removal and Granite Installation

During the seminar, the oral presentations by the speakers were recorded. Subsequently, the recorded oral presentations were transcribed. These written transcripts were edited to be presented in a written report type format. The edited transcripts were reviewed and approved by each speaker and were compiled together into this book, which represents the proceedings of the seminar.

Editor
Ian R. Chin
Exterior Wall Task Force Chairman
The Chicago Committee on High Rise Buildings

ACKNOWLEDGEMENTS

The Chicago Committee on High Rise Buildings wishes to gratefully acknowledge the help of the following in making this book possible: Amoco Corporation, the Building Owner; Rick Cantalupo, Ian R. Chin (Editor), John Dombrowski, Roger Hage, William Weis, Bernie Simmons and Jack Stecich, the Speakers; Brenda Garcia, Transcriber; Gina Montalbano, Manuscript Coordinator; and Wiss, Janney, Elstner Associates, Inc., Manuscript Preparation.

CHAPTER 1
INTRODUCTION
BY
IAN R. CHIN
Vice President and Principal
Wiss, Janney, Elstner Associates, Inc.

After the recladding of the Amoco building was completed, I approached AmProp Finance, Inc., (Amoco) on behalf of the Chicago Committee on High Rise Buildings and requested Amoco's approval and participation in presenting a seminar on the investigation, repair design, and recladding of the Amoco building. After considering our request, Amoco agreed to present and participate in the seminar because they wanted to present the lessons learned from this project to minimize the reoccurrence of the recladding experience on other buildings.

The Chicago Committee on High Rise Buildings, the sponsor of the seminar, was formed in 1969. It is an interdisciplinary group of architects, building owners and managers, construction managers, contractors, developers, engineers, equipment and material manufacturers and suppliers with significant expertise on high rise buildings. The purpose of the Committee is to initiate and support activities that will lead to the proper design, construction, management, and maintenance of high rise buildings; and to stimulate cooperation and innovation within the building industry. To achieve this purpose the Committee sponsors research and gathers, correlates, and disseminates information on high rise buildings to interested parties throughout the world.

The members of the Committee who have worked extremely hard to plan and present this seminar are:

John E. Fialkowski - Financial

Jon M. Boyd - Hotel and Symposium Facilities

Ferd Scheeler - Graphics

Duane Sohl - Printing and publications

Thomas J. Wysockey - Registration

To begin the seminar, Roger Hage, Vice President of AmProp Finance, Inc., will present information on the design and construction of the original marble panels on the building. He will also present information on the design of the building and on how the marble was selected. After Roger's presentation, Ian R. Chin, will make a presentation on the investigation that was undertaken to evaluate the condition of the original marble panels which resulted in the recommendation to remove the marble panels from the building. Roger will then present Amoco's response to this recommendation and will describe the various repair options studied.

The next session will be on the design of the new granite cladding and will be presented by John Dombrowski, Project Manager, of Amoco Properties, Inc., and Jack Stecich, Senior Consultant of Wiss, Janney, Elstner Associates, Inc.

Following this session, there will be three sessions on the removal of the original marble panels and on the construction of the new granite panels on the building by Bernie Simmons, Engineering Supervisor of Amoco Oil Company; Rick Cantalupo, General Superintendent of Schal Bovis, Inc.; and William R. Weis of W.R. Weis Company, Inc.

Roger Hage and Ian R. Chin will than present post construction conclusions, and summarize the lessons learned and the contributions to the building industry that were obtained from this project. All of the speakers will then assemble at the podium to answer questions from the audience.

So without any further ado, it is my great pleasure to introduce Roger Hage, Vice President of AmProp Finance, Inc. who will present information on the design of the building and of the marble panels on the building.

CHAPTER 2
DESIGN OF ORIGINAL MARBLE CLADDING
BY

ROGER HAGE
Vice President, AmProp Finance, Inc.

For those who do not know me, I am the Facility Manager for the AMOCO headquarters building at 200 East Randolph in Chicago, IL. I want to go through a couple of terminologies. We use the term AmProp Finance, Inc., who is technically the owner of the building. AMOCO Properties, Inc., is the building managing entity and building construction management department. The AMOCO Oil Company is the refining/marketing/operating company. All three of those entities are wholly owned subsidiaries of AMOCO Corporation. So in essence we all work for the corporation.

We have agreed, as Ian pointed out, to participate in this seminar. Basically, we think that this is information that should be divulged to everyone so that, as Ian said, this occurrence does not happen again. We are doing it also to improve the information on the construction of the high-rise building which is one of the charters of the Chicago Committee on High Rise Buildings. Today we will be dealing with the stone cladding experiences that we have had, both initially and the second time we did it.

We entered into this agreement that we would provide the information with two caveats: one being that we will not discuss the cost of the recladding project and the second one is that we will not talk about the legal issues involved with the cladding problem. We will make every effort throughout the presentation to discuss the technical facts associated with it and not the players that have been involved with it.

To start out, I would like to give you some basic information about the building. First the land was purchased in August, 1968. Ground breaking was April 6, 1970, and the first occupancy was December, 1972.

The building is situated on an entire city block. Basically, you think of it as an 80-story building. It has 80 usable stories from the Randolph Street level, as shown in Fig. 2-1. It is really two buildings in one. There is the tower section which is visible to everyone, and underlying the tower section is the podium building which extends five stories below Randolph Street and covers the entire city block.

The usable area of the building is 2,520,000 square feet, the rentable area is 2,848,000 square feet, and the gross area of the entire structure is 3,507,000 square feet, as shown in Fig. 2-2.

It is 1,136 feet in height from the Randolph Street level. The floor slab-to-slab distances are 12 feet 8 inches and the floor to ceiling heights are 8 feet 8 inches. It has a center core design with column-free space between the exterior wall and the center core, as shown in Fig. 2-3.

The use of the building: approximately 52 percent of the building (over time it fluctuates) is used by AMOCO Corporation to function as its international headquarters operation. Most operations in the building are headquarter functions.

The other 48 percent has been leased to outside tenants, such as, Kirkland Ellis, Price Waterhouse, Peterson & Ross, and the Mid-America Club. The needs of all of our tenants played an important role in the execution of this project.

Two things about the base structure. Fig. 2-4 is a photograph of an early model that was constructed and I took it out of the files because I think it is pretty descriptive. This model shows the exterior columns, the chevron columns on the outside which are now clad with granite, the spandrel beams, and the trusses that clear span the 45 feet from the exterior wall to the interior core. The trusses are spaced at ten feet on center and they are staggered 5 ft, 0 in. on every other floor.

Fig. 2-3 is a floor plan of one of the lower floors that also indicates the structure and the columns on the outside of the building. Terminology which you will hear throughout the presentation includes the term "reentrant corners," which are steel plate structures located at the four corners of the building.

At the interior of the building is the core, which contains five elevator banks, exit stairs, and mechanical shafts. Structurally the building is a tube construction which sits on 56 rock caissons. The elevator system was fairly unique at the time and still is pretty unique in that it serves all floors from the main building lobby. It is the largest double-decked elevator system installation in the world made up of five elevator banks, 40 passenger elevators, 80 elevator cabs. The speeds range from 900 feet per minute for the low-rise elevators, 1,600 feet per minute for the high-rise elevators.

The exterior wall was clad with 44,000 pieces of Alpha-Gray, which was the name of the marble from Carrera, Italy. The average piece size was 50 inches high by 42 inches wide by 1-1/4 inches thick, except for the reentrant corners where the thickness is 1-1/2 inches from about the 42nd floor and above. That was to compensate for the increased wind loads in those corners.

As you can see in Figs. 2-5 and 2-6, the marble panels were suspended from the columns through a series of galvanized angles and stainless steel bent-plate angles. The marble itself had kerfs cut into them 2 inches in, 7 inches up, and the next piece below was 7 inches down and repeated on the lower member and the upper member.

Each piece of marble was independently supported from a series of galvanized steel angles. The 5 by 3-1/2 inches by 1/4 by 4-3/4 inches long angle was stud-bolted into the structural steel columns. This angle supports a 5 by 3 by 1/4 by 6-1/4 long galvanized angle with slots that allow adjustment for up and down and in and out.

The 1/8-inch thick stainless steel bent-plate angles that support the marble pieces were connected to the 5 by 3 by 1/4 by 6-1/4 inch long angle. The downturned tabs of the bent plate angle that laterally support the marble piece below were placed in sealant filled kerfs in the panels, a bearing pad was installed between the bottom of the panels and the bent plate angle, and a compressible pad was installed between the top of the panels and the bottom of the bent plate angle. All of the joints between the marble panels were sealed with sealant.

Figures 2-7 and 2-8 are photographs of the marble installation. In these photographs, you can see the fireproof material in the space between the structural column and the marble. You can also see the galvanized steel angles, and the bent stainless steel angle with the tabs going in to the kerfs in marble panels.

Now in the beginning, some early testing that was done of the marble. The testing consisted not only of testing the stone, but there was a test on the suspension system, these were full-scale tests down in Florida for wind and water intrusion. I have some of the results here of the early testing that was performed.

In 1970, six marbles were tested. Three samples of each marble were tested. These were all Italian stones. For the first four stones, the test samples were cut from 2 ft by 2 ft panels. For the last two stones, the samples were cut from 1 ft by 1 ft panels. The average flexural strength from these tests showed that we were dealing with stones having 1,775 psi flexural strength. The probable minimum was 1,400 psi flexural strength. As shown in Table 2-1, the Haliso marble tested had a flexural strength ranging from 1,675 psi to 1,975 psi; the Vitro marble from 1,750 psi to 2,175 psi; the Carrera C marble from 1,450 psi to 1,575 psi; the Lavano marble from 1,300 psi to 1900 psi; the Alpha-Grey marble, which was the stone that was selected and put on the building, ranged from 1,475 psi to 1,950 psi; and the Edward 70 marble, which is the marble that we used in the interior cladding in our lobby, was 2,100 psi to 2,615 psi.

A second set of tests was performed by IIT Research to determine what the effect of thermal cycling would have on the marble. As shown in Table 2-2, the three stones that were tested for cycling were Alpha-Grey, again that is the stone that was placed on the building, the Edward 70, and a domestic marble. The Alpha-Grey had an initial average flexural strength of 1,861 psi when the face was in compression, 2,185 psi when the face was in tension, and after being subjected to 30 cycles of freeze/thaw from -10 degrees F to +120 degrees F, the average flexural strength decreased to 1,356 psi or a 38 percent reduction in strength. The Edward 70 was 2,368 psi in compression, and 2,117 psi in tension. It reduced to 1,469 psi, reflecting a 31 percent reduction in strength. There was also a domestic white marble that was tested, initially it had 1,191 psi in compression, 1,507 psi in tension. It reduced to 544 psi or a 64 percent reduction in strength through the thermal cycling.

As a result of these tests, Alpha Grey marble was selected for the marble on the building based on color. It was actually thought to be a white marble, but it was actually a gray marble that was selected specifically by the architect based upon its coloration and reflection of lighting effects, its structural strength, and its availability and delivery.

Now I will turn it over to Ian Chin. He will talk about the investigation that went on after the initial installation.

Early Testing

1970

6 Marbles Tested - 3 Samples Each

Average Flextural Strength	1775 psi
Probable Minimum	1400 psi

	Flextural Strength (psi)		
Helisso	1675	1775	1975
Vittro	1750	1775	2175
Cara C.	1450	1500	1575
Lavano	1300	1600	1900
Alpha Grey	1475	1775	1950
Edward 70	2100	2375	2615

Table. 2-1 - Results of preconstruction flexural strength tests performed on marble samples.

Early Testing

	Virgin	30 Cycles Freeze Thaw	
Stone	Average Flexural Strength (PSI)	Average Flexural Strength (PSI)	Loss %
Alpha Grey	1861 Face in Compression 2185 Face in Tension	1356	38
Edward 70	2368 Face in Compression 2117 Face in Tension	1469	31
Domestic White Marble	1191 Face in Compression 1507 Face in Tension	544	64

Table. 2-2 - Results of preconstruction flexural strength tests performed on marble samples before and after exposure to 30 cycles of thermal cycling.

Fig. 2-1 - View of Amoco building from the southeast soon after it was constructed in 1972.

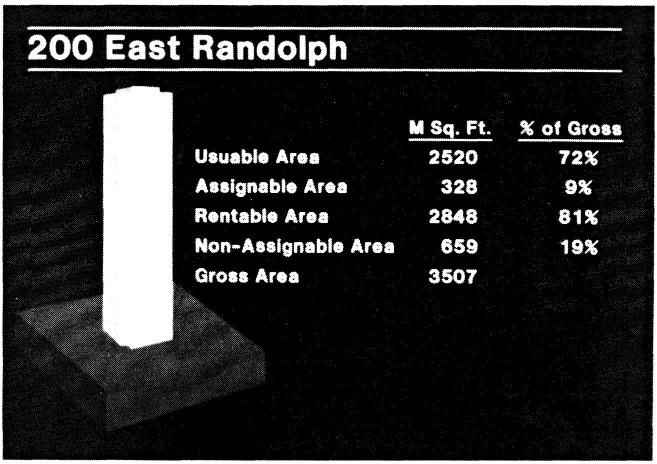

Fig. 2-2 - Basic useable area of building.

ZONE - A - FLOORS 3 TO 19

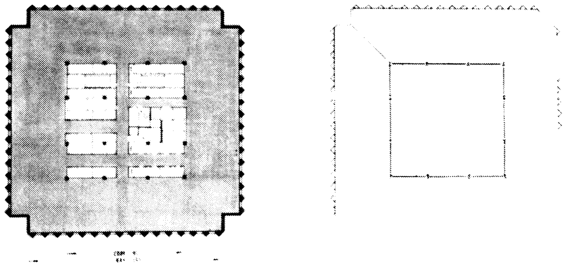

Fig. 2-3 - Typical core and structural framing plan of building.

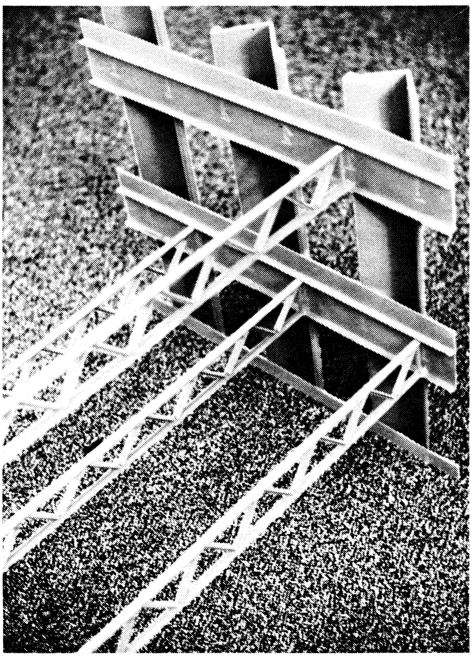

Fig. 2-4 - Model of structural frame of building showing the v-shaped (chevron) columns, spandrel beams, and floor trusses.

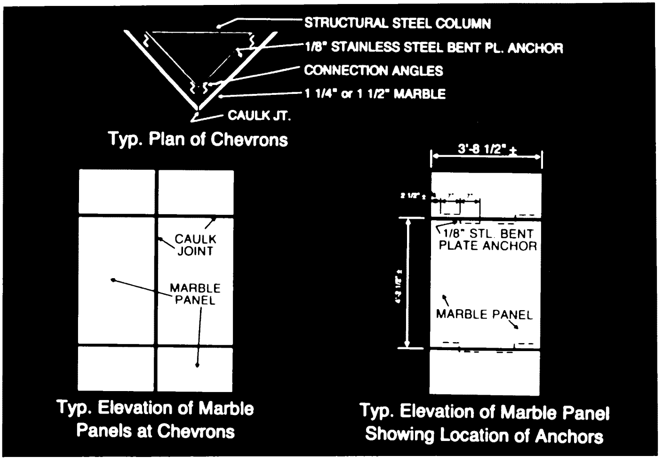

Fig. 2-5 - Typical details of marble panel connections.

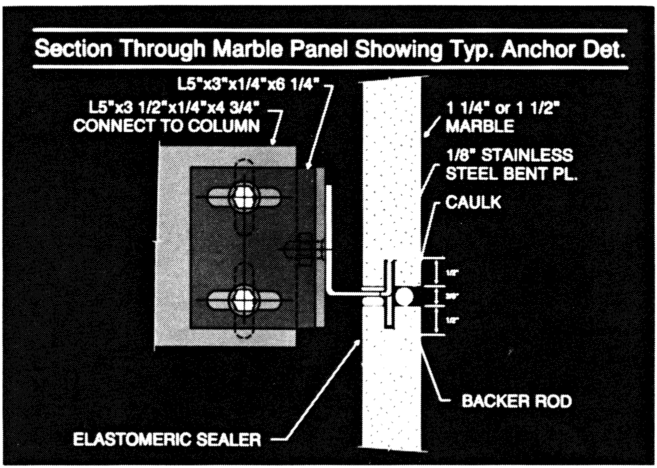

Fig. 2-6 - Section through marble panel connection.

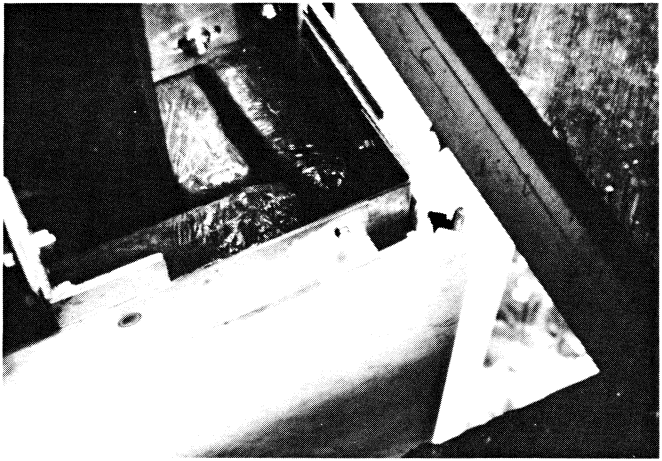

Fig. 2-7 - View of stainless steel bent plate angle that supported the marble panels.

Fig. 2-8 - View of stainless steel bent plate angle and steel column.

CHAPTER 3
INVESTIGATION OF ORIGINAL MARBLE PANELS ON BUILDING
BY
IAN R. CHIN
Vice President and Principal
Wiss, Janney, Elstner Associates ,Inc.

A photograph that was taken of the building shortly after it was constructed in 1972, is shown in Fig. 3-1. In this photograph the white marble panels gave the building a clean and distinctive appearance. After the building was constructed and occupied, the marble panels and windows on the building were regularly maintained by Amoco personnel in accordance with a maintenance program developed by Amoco.

The maintenance of the marble panels was performed from the building's suspended scaffold and generally consisted of close-up inspection of the panels, replacement of deteriorated sealant in joints, and repair of any cracks that developed in the panels. The repair of the cracks consisted of routing the cracks to create a cavity, routing transverse grooves across the cracks, drilling a hole at the ends of the routed joint and grooves, insertion of stainless steel horseshoe pins in the routed grooves to stitch the crack, and sealing the crack and grooves with epoxy.

1979 Inspection of Marble Panels

In 1979, Wiss, Janney, Elstner Associates, Inc. (WJE), was hired by Amoco to perform an inspection of the exterior facade on the building in accordance with the City of Chicago's building facade inspection ordinance. This ordinance, which was adopted by the City of Chicago in 1978, required the exterior facade on all buildings over five stories tall to be inspected close-up by a registered architect or structural engineer hired by the Owner of the building. After the inspection was completed, the inspecting architect or structural engineer was required to prepare a written report on the inspection that documented the condition of the facade, significant deterioration of the facade, evidence of observed movements of the facade, and the water tightness of the facade.

During this inspection, all of the marble panels on the building were inspected close-up and their condition was documented by architects and engineers from Amoco and WJE. This 1979 inspection revealed the following:

1. Crescent shaped cracks in approximately 230 marble panels at connections, as typically shown in Fig. 3-2. As Roger had previously described, the panels were laterally supported at their horizontal joints by the two, 6 in. long upturned and downturned tabs of the stainless steel shelf angle that were inserted into kerfs cut in the top and bottom of the panels. The observed crescent shaped cracks occurred in the panels at these lateral connection points.

2. Vertical, horizontal and/or diagonal cracks away from connections in approximately 1,800 marble panels, as typically shown in Fig. 3-3.

3. Outward displacement of approximately 1/2 in. in seven of the marble panels in the reentrant corners of the building, as typically shown in Fig. 3-4.

4. Elongation of some of the cracks in the panels that were previously repaired by Amoco, as typically shown in Fig. 3-5.

5. Outward bowing of some of the panels. The maximum outward bow observed was about 3/8 in.

The extent of the above distress in the panels observed in 1979 was not significant. As shown in Table 3-1, the 230 panels with crescent shaped cracks were 0.52 percent of the 44,000 panels on the building; the 1,800 panels with vertical, horizontal, and/or diagonal cracks were 4.3 percent of the panels; and the seven panels with outward displacement were 0.02 percent of the panels on the building.

After the 1979 inspection was completed, Amoco removed all of the seven panels with outward displacement, repaired the cracks in the panels, and continued to inspect and maintain the marble panels on the building.

<u>1985 Inspection of Marble Panels</u>

Between 1979 and 1985, Amoco personnel observed that additional cracking and bowing had occurred in many of the panels on the building, and that the cracking and bowing of panels were continuing to occur at an accelerated rate. As a result of this observation, Amoco hired WJE in 1985 to perform another close-up inspection of all of the marble panels on the building.

The 1985 inspection revealed the following:

1. Crescent shaped cracks in marble panels at connections in approximately 2,780 or in approximately 6 percent of the 44,000 panels on the building, as shown in Table 3-2.
Between 1979 and 1985, the number of panels with this type of cracking had increased by approximately 2,560 panels or by approximately 1,100 percent, as shown in Table 3-3. This type of cracking included panels with new, fine, vertical cracks at the connections that were observed for the first time by WJE in 1985. When these cracks were lightly tapped with a hammer they formed crescent shaped cracks, as shown in Fig. 3-6.

2. Vertical, horizontal, and/or diagonal cracks in marble panels away from connections in approximately 5,500 or in approximately 12 percent of the panels on the building.

Between 1979 and 1985, the number of panels with vertical, horizontal, and/or diagonal cracks increased by approximately 3,700 panels or by approximately 190 percent.

3. Typical outward bowing of marble panels of approximately 1/2 in. and at a maximum of 1-1/8 in., as shown in Fig. 3-7.

Between 1979 and 1985, the maximum bow in the panels increased by approximately 5/8 in. or by approximately 200 percent. The observed outward bowing of the panel was actually an outward dishing of the panels because the bow occurs along both the vertical and horizontal axis of the panels. Approximately 80 percent of the significantly bowed panels were located on the south and east elevations of the building, which unlike the west elevation, have no adjacent structures to block direct exposure to the sun. The marble panels on the north elevation of the building had the least amount of outward bowing.

Marbles primarily consist of calcite crystals. Calcite crystals are anisotropic. This means that when they are heated the crystals expand in different amounts in different directions, and when they are subsequently cooled, they cannot return to their original position because the crystals interlock. This phenomenon is called hysteresis, and it often results in a permanent expansion in the marble and an accompanying strength loss. Hysteresis also often causes marble panels to dish when one of the faces of the panels is heated to a higher temperature than the other, thus causing the side exposed to the higher

temperature to expand more than the other side. It was, therefore, not surprising to find that the vast majority of the significantly outwardly bowed panels were located on the sides of the building that were exposed to the divert rays of the sun, and that the panels with the least amount of outward bowing existed on the north side of the building that was least exposed to the divert rays of the sun.

Investigation of Marble Panels

The 1985 inspection by Amoco and WJE confirmed that cracking and bowing of the marble panels on the building were accelerating at a rapid rate. When this information was presented to Amoco, Amoco formed a team to perform an investigation to determine the cause(s) of the cracking, the effects of the cracking on the ability of the panels to support design loads, and to recommend remedial action. The team consisted of members of Amoco's architectural and engineering staff, architects and engineers from WJE, and John Logan a geological consultant with Amoco.

The investigation performed by WJE consisted of laboratory testing of 96 marble panels removed from the building, laboratory testing of 10 virgin original panels stored in the basement of the building, in-situ testing of 48 panels on the building, and structural analysis.

Laboratory Testing of Marble Panels

One of the objectives of the laboratory testing was to obtain a basis for estimating the present and future flexural strength of the marble. The laboratory tests were performed during the period of February, 1988, through December, 1988, and included the following:

1. Flexural and connection tests on full sized panels under uniform load.
2. Testing of panel connection kerfs.
3. ASTM C880 flexural strength tests on rectangular prisms cut from the panels.
4. ASTM C170 compressive strength tests on cores cut from the panels.
5. Permeability testing of marble samples cut from the panels.
6. ASTM C97 absorption and specific gravity tests.

7. Laboratory simulated accelerated weathering test on ASTM C880 rectangular prisms cut from the panels. This test consisted of submerging the outside 1/2 in. of the prisms in 0.01 molar solution of sulfurous acid to simulate acid rain and exposing the prisms in this condition to 100, 200, or 300 cycles of an air temperature range of -10 degrees F to 170 degrees F. Testing progress averaged about two cycles per day. This weathering test is currently being used by many designers as a part of the testing program for stone on buildings.

8. Petrographic examination to determine the mineral composition of the marble.

The laboratory tests revealed that the marble lost and would continue to lose significant strength due to its exposure to heating and cooling cycles, and that a marble panel with a large bow tends to have a lower flexural strength than a marble panel with a small bow. The laboratory tests also revealed that there was a strong relationship between the flexural strength of full sized panels and the flexural strength of ASTM C880 rectangular prisms cut from the panels.

Examination of the ASTM C880 flexural strength tests performed on the virgin original marble panels stored inside the building before and after exposure to the accelerated weathering test conditions revealed that 100 cycles of the laboratory simulated weathering are approximately equivalent to about 10 years weathering in Chicago, as shown in Fig. 3-8.

The results of the laboratory ASTM C880 flexural strength tests are summarized in Table No. 3-4, and revealed the following:

1. The average flexural strength of the virgin original marble panels was 1,260 psi.

2. After about 16 years on the building, the marble panels with average flexural strength lost about 40 percent of their original strength and had a flexural strength of 760 psi.

3. After another 10 years on the building, the marble panels with average flexural strength will lose a total of about 70 percent of their original strength and have a flexural strength of about 530 psi.

4. The minimum flexural strength of the virgin original marble panels was 1,090 psi.

5. After about 16 years on the building, the marble panels with minimum original flexural strength lost about 75 percent of their original strength and had a flexural strength of 285 psi.

6. After another 16 years on the building, the marble panels with minimum flexural strength will lose a total of about 85 percent of their original strength and have a flexural strength of about 170 psi.

As Roger previously mentioned, the design of the original marble panels on the building was based upon the panels having a minimum flexural strength of 1,400 psi and a maximum strength loss of 40 percent. Roger also mentioned that these design strength values were obtained from predesign laboratory tests performed on marble samples obtained from marble quarries.

The laboratory tests performed on the actual marble panels used on the building demonstrated that the marble supplied for the building did not conform with the project design requirements because its original flexural strength was 1,090 psi or about 22 percent less than the minimum design flexural strength of 1,400 psi, and because the actual strength loss of the marble supplied was about 75 percent or about two times more than the design strength loss of 40 percent. Due to this condition, under design wind load the factor of safety of some of the marble panels on the building was less than one.

Insitu Testing of 48 Marble Panels

Insitu load tests were performed on 48 marble panels on the building by WJE during the period of November, 1987, through April, 1988. The panels tested were selected based upon their location and extent of bowing.

The purpose of the insitu load tests was to assess the ability of the marble panels and their connections, in their condition at the time of the tests, to support negative wind load.

Prior to testing, the sealant in the joint around the perimeter of the panels was removed to eliminate possible contribution of the sealant to panel strength. A simulated uniform negative wind load was applied to the panels tested with hydraulic rams at the back of the panels, as shown in Fig. 3-9. The results of the insitu tests are summarized in Fig. 3-10 and are as follows:

1. The typical marble panels tested failed at equivalent negative uniform wind loads that varied from 45 psf to 100 psf.

2. Forty-four, or about 92 percent of the 48 marble panels tested, failed at their kerf connections. All but one of these 44 panels failed at their bottom connection suggesting that the sealant in the kerf at the top of the panels added strength to the connection.

3. Four, or about 8 percent of the 48 marble panels tested, failed on flexure.

4. No discernable relationship between panel connection strength and bow in the panels was found.

The insitu load tests revealed that under design wind loads the marble panels on the building had a safety factor of about one, and, therefore, did not have an adequate safety factor.

Recommendation

The results of the laboratory testing of 96 marble panels and of the insitu load tests of 48 marble panels revealed that the marble panels at the time of the tests could not support the design wind load with an adequate safety factor, and that with additional exposure to weathering, the marble panels will continue to loose strength and the situation will become worse. Based upon the test results, WJE made a recommendation to Amoco, on a preliminary basis, to remove all of the marble panels from the building. I will now let Roger tell you how he reacted to this recommendation.N.D.

TABLE 3-1 - SUMMARY OF 1979 INSPECTION OF MARBLE PANELS

CONDITION	NO. OF PANELS	PERCENT OF 44,000 PANELS
Crack in Panel at Connection	230	0.52
Crack in Panel Away From Connection	1,870	4.30
Outward Displacement	7	0.02
Outward Bow	3/8 in. Max.	---

TABLE 3-2 - SUMMARY OF 1985 INSPECTION OF MARBLE PANELS

CONDITION OF PANELS	NO. OF PANELS	PERCENT OF 44,000 PANELS
Crack in Panel at Connection	2,780	6.30
Crack in Panel Away from Connection	5,440	12.40
Outward Displacement	0	0
Outward Bow	1-1/8 in. Max.	80 Percent of Bowed Panels were on South and East Elevations

TABLE 3-3 - COMPARISON OF CONDITION OF MARBLE PANELS BETWEEN 1979 AND 1985

CONDITION	1979	1985	INCREASE
Crack in Panel at Anchor	230	2,780	1,100% ±
Crack in Panel Away from Anchor	1,870	5,440	190% ±
Outward Bow in Panel	Up to 3/8 In.	Up to 1-1/8 In.	200% ±

TABLE 3-4- FLEXURAL STRENGTH OF MARBLE PANELS

CONDITION	DESIGN	ACTUAL	DIFFERENCE
Original Strength	1,400 psi Min.	1,260 psi Avg.	---
		1,090 psi Min.	22% Less Than Specified
After Aging	840 psi (40% Loss)	530 psi Avg.	---
		170 psi Min. (90% Loss)	80% Less Than Specified

Fig. 3-1 - View of Amoco building shortly after it was constructed in 1972.

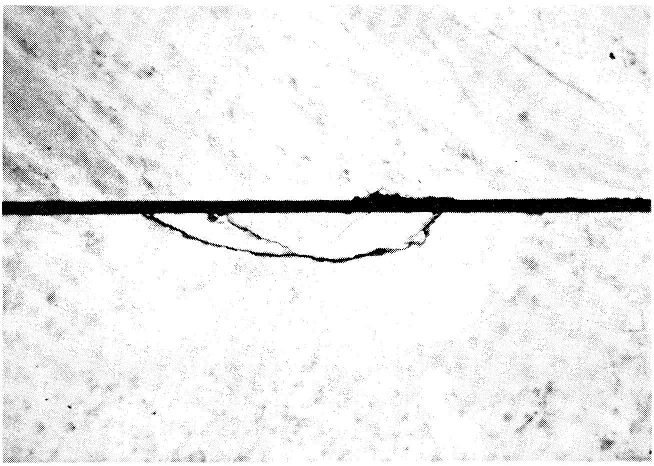

Fig. 3-2 - View of typical crescent shaped crack in marble panel at connection.

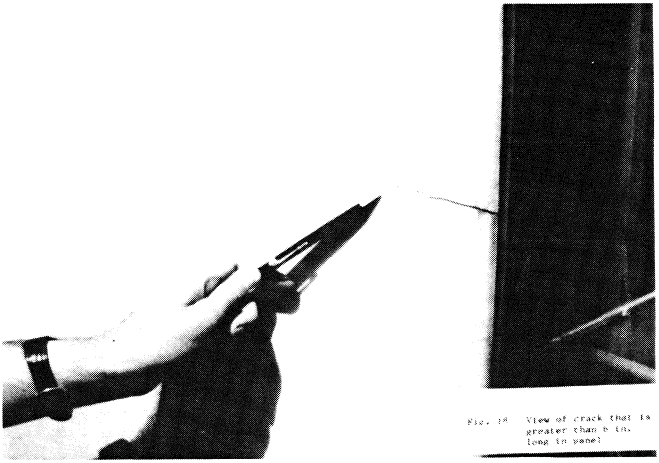

Fig. 3-3 - View of diagonal crack in marble panel away from connections.

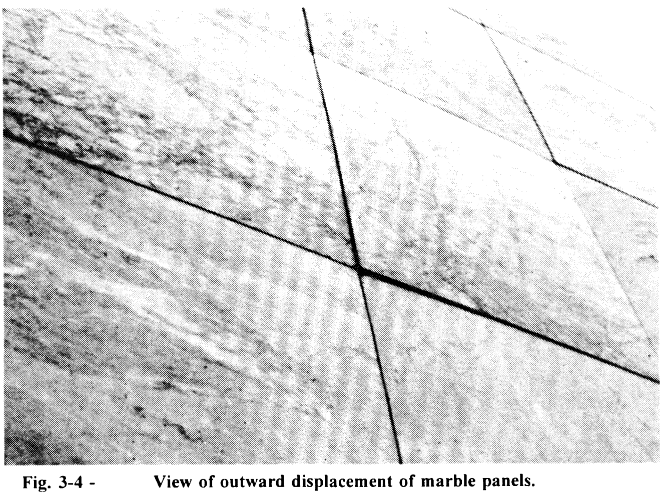

Fig. 3-4 - View of outward displacement of marble panels.

Fig. 3-5 - View of a previously repaired crack that increased in length.

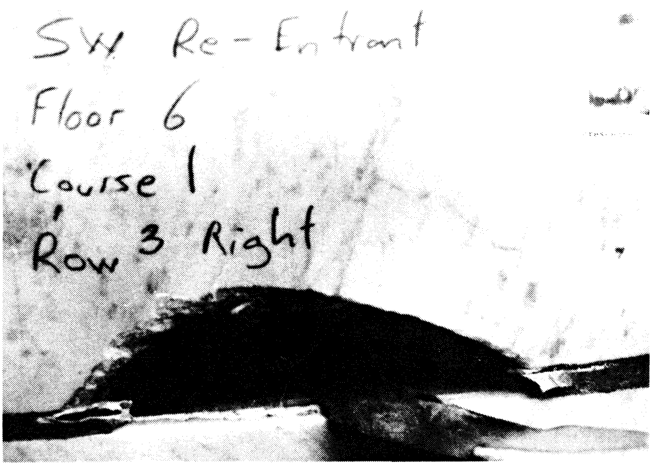

Fig. 3-6 - View of crescent shaped crack and fracture that occurred in marble panels after fine, vertical cracks in the panels at connections were lightly tapped.

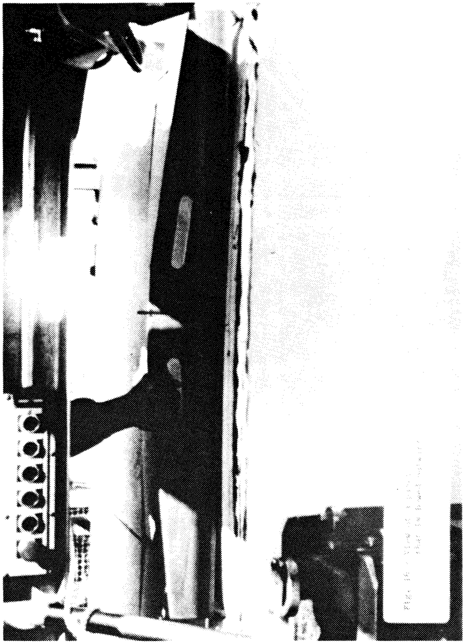

Fig. 3-7 - View of a marble panel that is bowed outward.

C880 STRENGTH vs. YEARS OF EXPOSURE

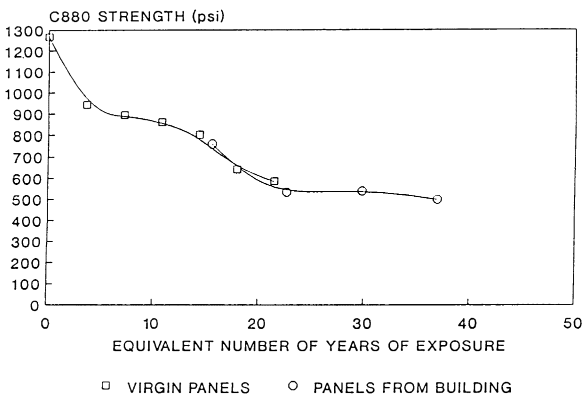

Fig. 3-8 - **C880 strength - 15 years of exposure. One hundred cycles of laboratory simulated weathering are approximately equivalent to about 10 years.**

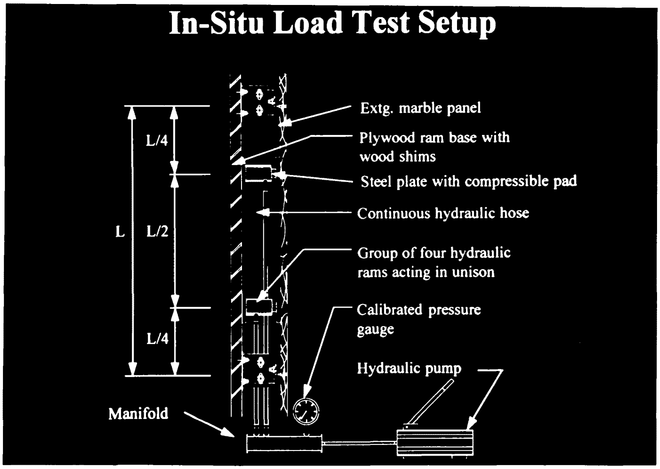

Fig. 3-9 - Set-up on insitu load test of marble panels.

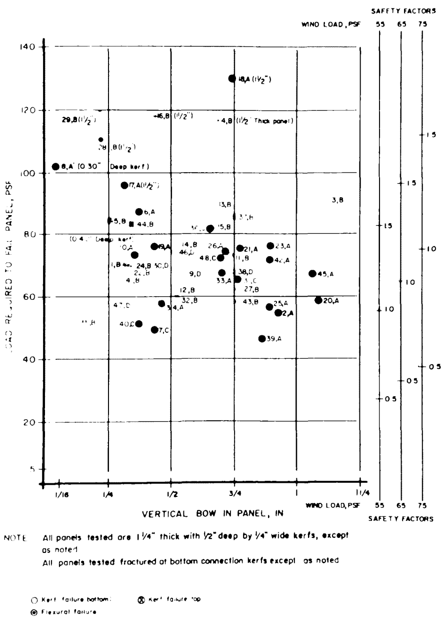

Fig. 3-10 - Summary of insitu load tests.

CHAPTER 4
REPAIR OPTIONS
BY
ROGER HAGE
Vice President, AmProp Finance, Inc.

When a person comes in and says 44,000 pieces of marble on your building have to be removed, you go "whomp!". Well, after I picked myself up off the floor, we said there has got to be some other solution besides removing the marble. We looked at about six or seven different options of what we could possibly do and this again was by the team that Ian had described that was working together to address this situation.

As shown in Fig. 4-1, the first idea was to cut and pin the existing marble. We believed that if the marble had lost strength, we could cut a slab of marble vertically in half and horizontally in half, and we would make four bolted connections for each section of the panel to reduce its span and pin each section back to the structural steel, caulk it up, put plugs over it, and away we go. Granted, this option was not going to result in the most beautiful looking building after you get through doing that. We looked at this option and estimated that it would cost about $50 million and it would take about five years to complete. During this time, the strength loss curve that Ian described was continuing to go down. We did not even see it bottoming out, so we were a little reluctant to go ahead with this repair option. In addition, the concern of making a blind connection as you drill and tap into the steel without knowing whether or not you have made a really solid connection, and if you take 44,000 pieces of marble and you put in 16 bolts per piece, you are talking about an awfully lot of connections. Then you have to maintain these connections. We would also have to maintain four times as much caulk as we had on the building before. When we put these conditions and concerns together, we felt that this option was not a recommended alternative.

So then we said, why do we not go along and set up a criteria to measure the strength of the marble and when it decreased to a point where we know it is no longer safe or when the bow has gotten such that we may shatter some of the kerfs, we take those pieces off and set up some kind of system where

each year we will survey the building, make a determination of how many panels have to be replaced, and then we will go up and replace them as needed. We would replace them with a new, thicker marble that is 1-1/2 inches thick. This option was estimated to cost about $57.4 million and would take about eight years. Again, because we were dealing with marble that was continuing to loose strength and had a lot of unknowns and the cost to do that and the disruption involved for eight years, we did not feel that this option was a recommended solution.

We considered replacing the marble with a 2-1/2-inch thick marble. Pretty much the same scenario, $72 million. Replace it with 1-1/2-inch marble but add some additional bolted connections to it. Again, we did not think that was an acceptable solution.

We then looked at replacing the marble on an as-needed basis with a new 2-1/2-inch marble. We figured that this option would take 20 to 22 years to complete and that it really was not practical to have what we call a world-class building, first class space with a scaffold hanging over it for 20 years. We did not think that was a good solution.

It really boiled down to the last three: Completely, systematically remove the marble and replace it either with a granite or aluminum skin.

After we decided on a preliminary basis to replace the skin, we mobilized a team. Management appointed Shelby Pierce, our General Manager of Engineering of the AMOCO Oil Refining and Transportation Engineering Department, as the project director. He was selected as the project director because he is the senior engineer within the Corporation. That pretty much tells you the importance that the Corporation placed on this project.

Project management was provided by the AMOCO Oil Company engineering department. The team was made up of Shelby Pierce, Steve Holdaway, Bernie Simmons, John Dombrowski, and Jim Courtney.

The next decision we made was to retain Wiss, Janney, Elstner Associates, Inc. to continue as the architect and engineer of record for this project. The construction management contract was the next one that we took into consideration. On December 18, 1987, we interviewed a number of construction managers

and told them the problem at that point and that this was not public knowledge. We asked them to keep it quiet. We interviewed the construction managers. We asked them to submit a proposal in ten days, and we awarded the project on January 4, 1988, to Schal Bovis, Inc.

The project was put into two phases. They were to work as part of the team and come up with solutions for the immediate stabilization of the existing facade and then work towards a removal and replacement strategy for the long term.

John Logan, who Ian has mentioned, had worked with us since early 1984 or 1985. John came to us as a recommendation from our Exploration Department as an expert in rock mechanics. He had previously worked with stone and on rock behavior. He was retained by AMOCO to do our quarry testing program for the new granite that we would use on the building.

Voy Madeyski was hired as our aesthetic design consultant. The original concept, or the concept that we followed, was that we would not change the basic aesthetic design of the building. This was going to be a restoration of the structure and we would not change any of the design concepts of the original architects. Voy was hired and retained to handle of aesthetic questions that came up in conjunction with that strategy. Voy was also the aesthetic designer of the construction canopy, which we wanted to work as a first-rate canopy. Incidentally, Voy did continue with us on the project and completed our design for the Plaza.

Don Shorts was the stone consultant who gave us advice and counsel on the availability of stones from around the world.

Wind tunnel testing laboratories that were involved were Cermak Peterka Peterson, Inc., and RWDI. There were peer reviewers set up to analyze the results of those tests; CDC performed that function. We hired another testing laboratory to review and to look at the testing procedures that were going to be followed by Wiss, Janney, Elstner Associates, Inc., and to be sure that their procedures were correct. This is a little bit of belt and suspenders, but I think you agree that when you do this twice, you do not want to do it three times, so we did extra extra. Corrosion potential was analyzed by John Slater.

I would like to mention too that during this process, we were dealing with the City of Chicago Building Department, who had a very strong interest in this project, as we did. The City Building Department worked with our team very closely. We were able to give them the information that they required and we were able to meet certain dates that were required, and it was a great partnership that we worked out with the City Building Department. It was really a good example of a professional working relationship with the City. We have nothing but praise for the City on this project. They did an outstanding job of helping us through this situation.

The stone was provided by the North Carolina Granite Corporation. It is Mount Airy granite. Stone removal and installation was handled by SESCO, Inc. The electrical subcontractor was CLC. The caulking was done by Riggio Companies. The hoisting, which was a monumental task, was handled by USA Hoist Corporation. Insulation was by Insulation Systems, Inc.

Some of the other concerns that we had in starting this project and developing teams, were:

1. Our tenants: How do we run a first-class building, which we built as a world-class building, getting high-end market rates of rent from these people in the building, provide the normal first-class service that we always have provided to them, and yet have major construction on the outside of the building? We made this process simple. We set up a strategy to completely segregate the two operations. Then we turned it over to the project team and said, "Handle that for us." You will hear how they did that.

2. Public Relations: How do we handle public relations? How do we release this information without causing panic and overdo concern? We made a field trip out to Chase Lincoln Bank in Rochester, New York. Some of you may know that they had a similar problem where they had to replace the marble cladding on their building, and we asked them to share their experiences. Probably the most important thing that they told us was to be upfront, and be honest and forthright with our tenants, the media, and with the public and you probably will not have a problem. So we worked with our public relations department and we utilized that strategy, being upfront and forthright, and it worked beautifully. I think that if you remember we had very little adverse publicity on this. It was very positive and that would be another lesson learned if any of you has the misfortune of having to go through something like this. Tell the truth.

Now what I would like to do is to hand the next session off to the members of the project team who will take you through the rest of the presentation. I would like to introduce John Dombrowski who was the project architect for Amoco Properties, Inc., and Jack Stecich, Senior Consultant with Wiss, Janney, Elstner, Associates, Inc. Their part of the talk is going to be about the design of the new granite cladding.

Solutions Studied

		Estimated Cost	Time for Completion	Feasibility	Recommended Alternative
1	Cutting & Pin Existing Marble	50 MM	5 Years	Unacceptable Due to Marble Strength	No
2A	Replace Marble as Needed with New 1½" Thick Marble	57.4 MM	8 Years	Unaccepatble-Engineering Requirements for Marble Strength and Quality Control	No
2B	Replace Marble on a Systematic Program with New 2½" Thick Marble	72 MM	7 Years	Unacceptable-Quality Control, Additional Weight to Structure, Cost Effectiveness Compared to Other Alternatives.	No
2C	Replace Marble as Needed with New 1½" Thick Marble with Additional Bolted Connections	64 MM	10 Years	Unacceptable-Engineering Requirements for Marble Strength and Quality Control	No
2D-2F	Replace Marble as Needed with New 2½"	80 MM - 88 MM	20 - 22 Years	Unacceptable-Quality Control, Additional Weight to Structure, Time for Completion	No
3	Complete Systematic Replacement with Granite	75 MM	4 Years	Good	Yes
4	Complete Sytematic Replacement with Aluminum	60 MM	3 Years	Good	Yes

Fig. 4-1 - Summary of solutions studied to address condition of marble panels.

CHAPTER 5
STABILIZATION OF EXISTING MARBLE PANELS AND PRELIMINARY SELECTION OF RECLADDING MATERIALS

BY
JOHN DOMBROWSKI
Project Manager
Amoco Properties, Inc.

Thank you Roger and Ian.

After the decision was made to take off the marble, we had to design the new cladding system. But before we could really begin the design, we had to look at securing the existing marble panels on the building. The first step for the project team was to secure the marble panels which could possibly fall due to failure under wind load.

Since the recladding could not actually begin for a year to a year-and-a-half, and with some panels at the top of the building not being able to be replaced for another year-and-a-half after that, Amoco felt that this exposure was too great and, therefore, a mechanism for securing the panels back to the building was our first concern.

The project team evaluated various solutions for restraining the marble panels such as bolting each of the corners, netting the panels, and even epoxying the face of the panels to get additional strength. These alternatives did not meet the criteria that the restraining system needed, that is, the panels needed to be restrained from falling off the building if it failed through flexure, or at their connections. The restraining system also needed to allow the marble panels to continue to bow and not allow additional stresses to build up, as well as have a positive connection back to the connection angles.

The solution of a belt and suspenders approach was adopted, and a stainless steel strap was bolted through the marble into the supporting angles, as shown in Figs. 5-1 and 5-2. The bolts were hand-tightened to allow the stainless steel strap to continue to expand and contract while also allowing the marble to continue to bow. The bolt had a nylock patch on the threads to prevent the bolt from backing out of the connection. The straps were powder coated with a color to match the marble for aesthetic reasons. Before

strapping could commence, the suspended scaffolds needed to be designed and constructed to perform the work.

The strapping of the marble panels started on April 5, 1988 and was completed on December 15, 1988. A total of 88,000 straps and 88,000 bolts were used in this operation.

While the strapping installation and final marble testing were proceeding, the project team also was investigating alternate solutions for recladding material. In all, over 50 materials were analyzed.

First, the feasibility of changing the overall appearance of the Amoco Building by the use of materials such as stainless steel, glass, limestone, or metal panels other than white were considered. After this review, the consensus was to try to maintain the existing image of the building. By this decision, the alternatives were reduced to white granite panels, white aluminum panels, and a man-made ceramic glass material.

A vigorous testing program was then developed to determine the response of recladding materials to the following criteria: flexural strength, consistency, absorption, and accelerated aging.

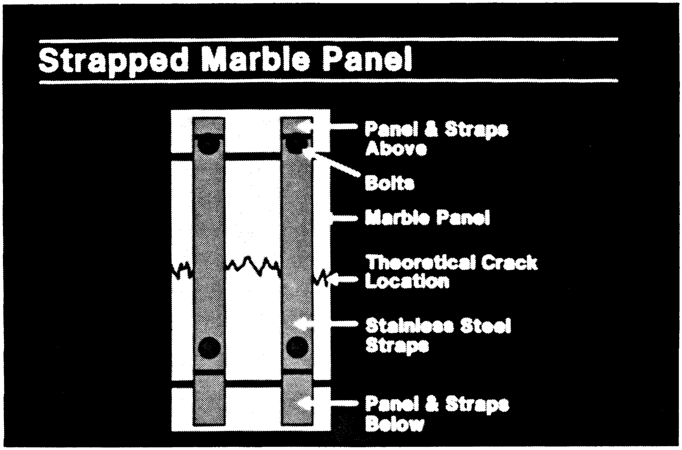

Fig. 5-1 - Elevation of straps used to temporarily restrain the marble panels.

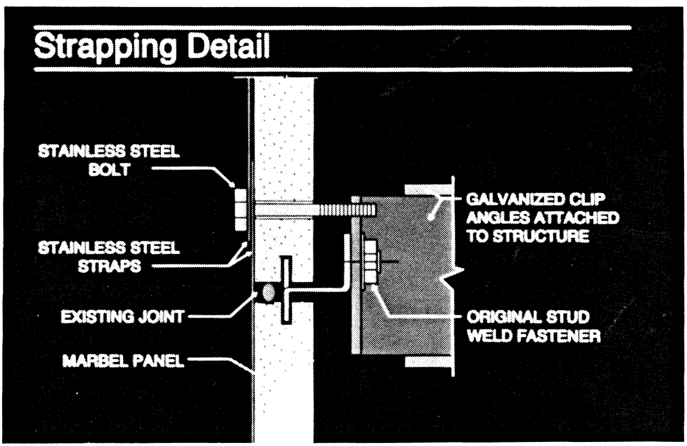

Fig. 5-2 - Typical section through straps.

CHAPTER 6
TESTING OF GRANITE SELECTED FOR RECLADDING THE BUILDING
BY

JACK STECICH
Senior Consultant
Wiss, Janney, Elstner Associates, Inc.

The evaluation of various types of stone being considered for recladding the building was done using laboratory testing that included standard ASTM flexural strength, modulus of rupture, compressive strength, absorption, and bulk specific gravity tests. The flexural strength testing on some of the samples was performed both as received from the quarry and after accelerated aging.

Fig. 6-1 is a view of C880 flexural strength testing being performed on a sample of granite. The sample is 2 in. thick, 3 in. wide, and 22 in. long. The sample is loaded at its quarter points, and the load is increased until the sample fractures. The stress that caused the sample to fracture is defined as the ASTM C880 flexural strength.

By comparison, the ASTM C99 modulus of rupture test on a granite sample is performed by placing a single point load at the mid-span of the sample of stone which is 2-1/4 inches thick, 4 inches wide, and 8 inches long, as shown in Fig. 6-2. You can see that this sample is shorter than the ASTM C880 sample. The ASTM C880 testing is a better representation of what occurs in a panel on a building.

The results of initial ASTM C99 modulus of rupture and ASTM C880 flexural strength testing for three of the stones that were considered are shown in Table 6-1. The samples tested were cut in two perpendicular directions from panels of these granites.

ASTM C99 modulus of rupture of Mount Airy granite was 1,750 psi in the stronger direction and 1,730 psi in the weaker direction, which is about the same in either direction. The ratio of wet strength to dry strength was 95 percent. One of the domestic granites tested had a modulus of rupture in the stronger direction of 2,000 psi and 1,560 psi in the weaker direction. The ratio of wet strength to dry strength was 92 percent. One of the European granites tested had a modulus of rupture of 1710 psi in one direction; test results in the perpendicular direction were not available. The ASTM C99 modulus of rupture values of all of the granites tested met ASTM minimum requirements.

The lower table shows ASTM C880 flexural strength for Mount Airy granite and for the domestic granite. The ASTM C880 flexural strength, as expected, is somewhat lower than the modulus of rupture. Again, the flexural strength of Mount Airy granite is about the same in two directions: 1470 psi in the stronger direction and 1,400 psi in the weaker direction. The flexural strength of the other domestic granite was 1,610 psi in the stronger direction and 1,290 psi in the weaker direction. The minimum C880 flexural strength specified by ASTM for granite is 1,200 psi. Both Mount Airy and the domestic granite met this requirement.

We also found that flexural strength for these granites would be reduced by about 10 percent after accelerated aging. The compressive strength, density, and absorption all met the ASTM requirements.

TABLE 6-1 INITIAL TESTING OF STONE

MODULUS OF RUPTURE

	STRONGER DIRECTION	WEAKER DIRECTION	WET to DRY RATIO
MOUNT AIRY	1750 psi	1730 psi	0.95
DOMESTIC GRANITE	2000 psi	1560 psi	0.92
EUROPEAN GRANITE	1710 psi	-	-

C880 FLEXURAL STRENGTH

	STRONGER DIRECTION	WEAKER DIRECTION	WET to DRY RATIO
MOUNT AIRY	1470 psi	1400 psi	0.92
DOMESTIC GRANITE	1610 psi	1290 psi	0.90

FLEXURAL STRENGTH REDUCED BY 10 percent AFTER ACCELERATED AGING

COMPRESSIVE STRENGTH, DENSITY AND ABSORPTION ALL MEET ASTM REQUIREMENTS

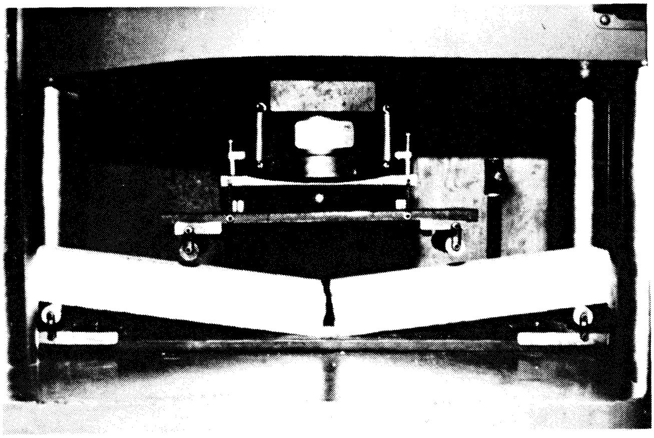

Fig. 6-1 - ASTM C880 Flexural Strength Testing of Granite.

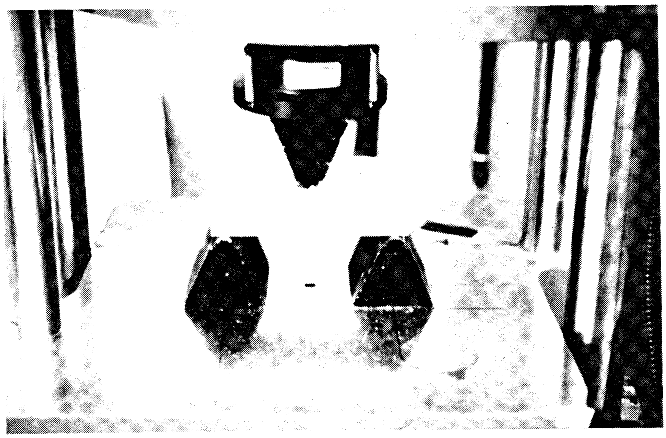

Fig. 6-2 - ASTM C99 Modulus of Rupture Testing of Granite.

CHAPTER 7
AESTHETIC MOCK-UP OF POTENTIAL RECLADDING MATERIALS
BY
JOHN DOMBROWSKI
Project Manager, Amoco Properties, Inc.

In addition to the testing program, other factors were evaluated which affected the final decision on which materials should be used to reclad the building. These factors were appearance, cost, availability and scheduling, sound qualities in particular to metal panels, finishes, repairs of the materials in the field, and technological limitations.

Although all the initial materials being considered were acceptable, the ceramic glass material was eliminated from further consideration because at that time this material did not have an established track record in the United States as a cladding material on high rise buildings. Therefore, the list was narrowed to four specific materials: two granites and two types of aluminum panels.

A full-scale mockup of a portion of the building consisting of a 3-bay-wide, 2-story frame was constructed at Amoco's Whiting Refinery in Indiana, as shown in Fig. 7-1. The mockup was clad with each material, one in each elevation, in the configuration as it would appear on the building. Observations were noted on the handling, the backup preparation, and the installation of each material. This was helpful in our final design.

The aluminum alternatives were a honeycomb panel and a form-bent aluminum panel. The granite selections were a domestic white granite and the Mount Airy granite quarried in North Carolina.

After a thorough review of the listed criteria, the final selection was the Mount Airy granite. Now you might say that since the material was selected, let's buy it and put it up. Well, this story is only one piece of the puzzle. The second question was "What was the stone going to sit on, that is, the shelf angle."

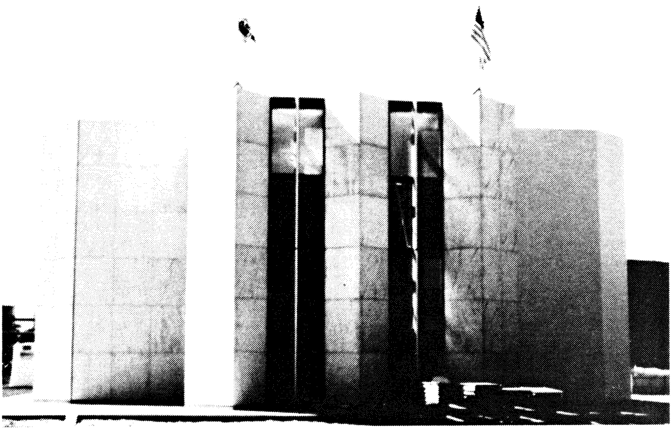

Fig. 7-1 - **View of full scale aesthetic mockup used to evaluate potential cladding materials.**

CHAPTER 8
STAINLESS STEEL VS ALUMINUM SHELF ANGLES
BY
JACK STECICH
Senior Consultant, Wiss, Janney, Elstner Associates, Inc.

Shelf angles are structural members that support the granite panels on the building. Both stainless steel and aluminum shelf angles were given very thorough consideration during the design of the new granite cladding system on the building. Stainless steel has a higher strength than aluminum, which required that aluminum shelf angles be thicker than stainless steel shelf angles.

Corrosion issues, including contact between dissimilar metals, were carefully evaluated. Since Amoco wished to save and reuse as many of the original support components as possible, the existing galvanized clip angles would be in contact with the shelf angle. Stainless steel has a lesser rate of dissimilar metal corrosion when in contact with galvanized steel than does aluminum. Even though the shelf angle is located behind the stone panels, chloride from salting of the street can penetrate into the cavity, particularly at the lower floors of the building, and cause corrosion. Aluminum is more susceptible to corrosion in the presence of wetting by chloride-laden water than stainless steel. The shelf angle sitting inside the kerf in the stone creates a crevice condition. Crevice corrosion is a greater concern in aluminum than in stainless steel.

Because the wind load pulsates, the shelf angles will be subjected to fatigue loading and the fatigue strength of stainless steel is about three times higher than the fatigue strength of aluminum.

Stainless steel shelf angles are more expensive and less available than aluminum shelf angles.

After considering all of these factors, stainless steel was chosen as the shelf angle material.

CHAPTER 9
SELECTION OF SEALANT AND WIND TUNNEL STUDIES OF BUILDING
BY

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Project Manager, Amoco Properties, Inc.

Selection of Sealant

After the selection of the recladding material and of the shelf angle material, a major component that needed to be evaluated was the type of sealant to be used in the joints between stone panels. Upon review of the Whiting mockup, five criteria were established for the evaluation of two different kinds of sealants, silicone and polyurethane. The criteria were adhesion, cohesion, staining of stone, dirt accumulation on the sealant, and life expectancy.

Initial sealant tests were performed on samples of stone in the laboratory. Six different sealants were tested. The test specimens were placed on the roof of WJE's laboratory in Northbrook, IL, and aged for approximately 18 months. The visual results are shown in Figs. 9-1 and 9-2. On the silicones sealants, there was considerable dirt accumulation as well as a staining problem on the edges of the stone.

A decision on the type of sealant to be used would have been made at this point based upon the initial testing. However, Amoco Properties Maintenance Department had concerns on how well the sealant would perform on the building. Therefore, another mockup consisting of five chevrons, two panels high, was constructed at the second floor of the Amoco building. Five manufacturers were invited to personally install their sealant on the mockup. Priming of the joints was at the discretion of each manufacturer.

Figs. 9-3 shows a portion of the sealant mockup on the building soon after its completion. Panels 1, 3, and 4 were silicones, Panels 2 and 5 were polyurethanes. Approximately three months after installation, we sprayed each joint with water to see if there was any evidence of staining from absorption of sealant into the stone. Staining on the stone would be evident as a dry band adjacent to the joints caused by the penetration of oils into the stone which basically waterproofed the stone. Up close visual inspection revealed that the stone panels with silicone sealants did have a horizontal band at the edge of the joints when wet, whereas stone panels with polyurethane sealant did not show any evidence of similar staining.

Approximately one year after the installation of the mockup, the sealant was reviewed again to detect any defects. The staining of the Mount Airy stone by silicone sealants was prevalent, as typically

shown in Fig. 9-4. The stone panels with polyurethane sealants did not show any evidence of staining. Upon evaluation of this mockup and the other criteria mentioned, polyurethane sealant was selected.

Wind Tunnel Testing

During the strapping design, as well as during the sealant selection, the project team was also focusing its attention on the wind loads to be used to design the new granite cladding. To determine the design wind load, a wind tunnel test was performed at Cermak Paterka Peterson in Fort Collins, Colorado, on a scale model of the building. A second wind tunnel test was performed by Rowan, Williams, Davies and Irwin in Ontario, Canada. We had two wind tunnel tests performed to obtain information on the design wind load.

Both tests revealed that the maximum design wind pressure on the cladding on the building was negative 75 psf for a 100-year wind. This maximum design wind load occurred in the mid to upper regions on the west elevation and in the northwest reentrant corner of the building and is higher than the design wind load specified by the current Chicago Building Code and by the Chicago Building Code in effect when the building was originally designed.

The most stringent wind loads were used for the design of the new cladding as well as to check the building's structural response to the weight of the new facade. First, the weight of the new facade had to be calculated. The design thickness of the granite was two inches thick, a considerable increase over the thickness of the original marble at 1-1/4 inches. In addition, the density of the granite was about 10 percent more than the original marble. Due to these conditions, the new granite panels were about 60 percent heavier than the original marble panels. A typical granite panel weighs about 450 pounds versus 270 pounds for a typical marble panel. This difference in weight resulted in an increase in cladding weight of about 3,750 tons, which is about one percent of the total weight of the building.

A detailed inspection was undertaken by WJE in representative areas on 15 typical floors to determine the actual weight of mechanical, electrical, and ceiling elements to be used in the design. With these loads now determined, the overall structure could be evaluated.

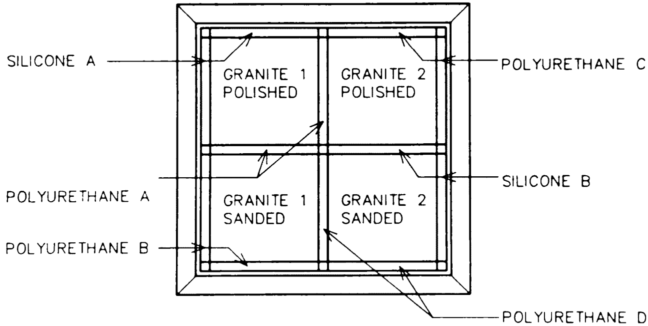

Fig. 9-1 - **Schematic elevation of sealant test panel.**

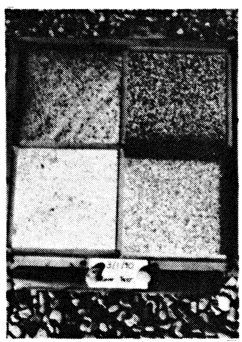

Fig. 9-2 - **View of sealant test panel after 18 months of outside exposure. Note staining of stone by silicone sealants and dirt accumulation on silicone sealants.**

Fig. 9-3 - **View of a portion of the sealant mockup on the building.**

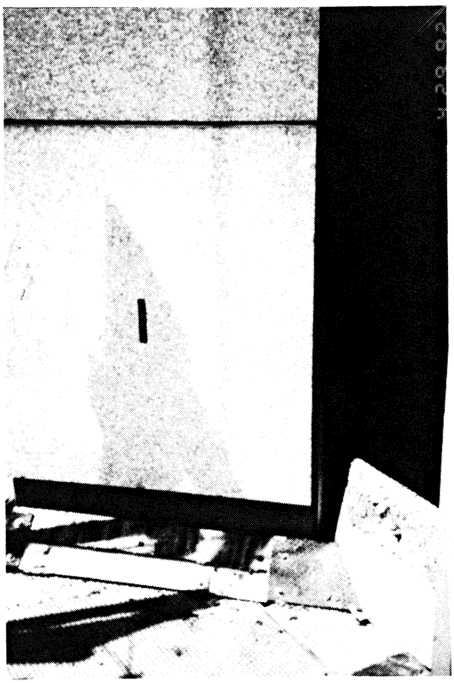

Fig. 9-4 - **Typical view of staining of granite adjacent to joints sealed with silicone sealants.**

CHAPTER 10
STRUCTURAL ANALYSIS OF STRUCTURE OF BUILDING UNDER INCREASED CLADDING WEIGHT AND INCREASED WIND LOADS

BY

JACK STECICH

Senior Consultant, Wiss, Janney, Elstner Associates, Inc.

The structural analysis of the building was performed to evaluate the effects of the increased cladding weight and increased wind loads that John Dombrowski described in Chapter 9. Wind loads specified by the current City of Chicago Building code are higher than wind loads specified at the time the building was designed. Another important issue in the structural review is improved computer capabilities. At the time the Amoco Building was designed, in the late 1960s, the structural analysis was performed on a mainframe computer at the Massachusetts Institute of Technology. Using state-of-the-art computing at that time, the designers were able to model only one-quarter of the building. There is a transfer girder at the LL1 level on the north face of the building that causes columns on the north face to have different loads than columns on the other three faces. This transfer girder was not included in the original model because of the need to use a quarter model with symmetry. Because of improved computer technology, we were able to model the entire building, interestingly enough, using a desktop computer.

After all of these issues were considered, seven columns at the lower levels on the north face, where the transfer girder is located, were found to have a slight overstress when the 100-year wind occurred. It was decided that these seven columns would be reinforced.

The columns were reinforced by welding new steel plates onto the existing steel columns after concrete fireproofing was removed. These steel plates were designed to bear directly onto the base plates at the bottom of the columns or onto the cap plate at the top of the column. A portable end mill was used to machine the base plates, as shown in Fig. 10-1, to provide a smooth, flat surface. The reinforcement plates were placed against the face of the column and brought into tight contact, and then were set to bear down on the machined surface of the base plate.

A similar machining operation was done at the top of the column where the column tops required reinforcement. The reinforcement plate was preloaded to hold it in firm contact with the cap plate above. The sides of the steel plates were then welded to the existing columns, as shown in Fig. 10-2.

The existing steel plate columns were up to 2-1/2 inches thick, therefore the American Welding Society (AWS) D1.1 welding code requires that the steel plate be preheated. Preheating was done to 225 degrees F and was limited to the area being welded to avoid expanding too great a length of the column. Because this was an occupied building, smoke that occurred from the welding operation was controlled with a "smoke-eater". The smoke-eater sucks in smoke from the area of the welding, pumps it to a filter, and filters the smoke.

Following reinforcement, the concrete fireproofing was placed back onto the columns.

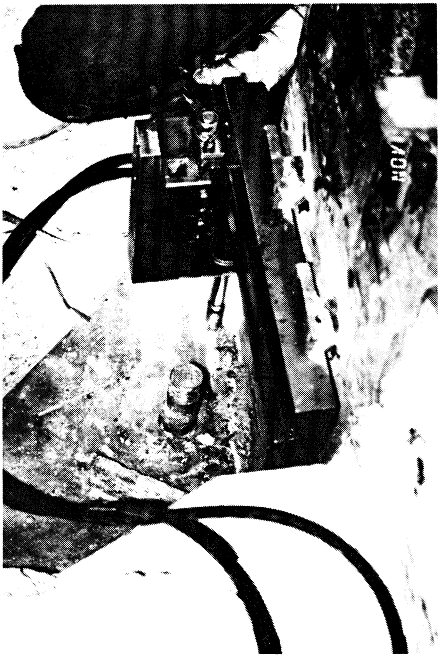

Fig. 10-1 - Portable end mill used to machine smooth, flat surface to base plate of column.

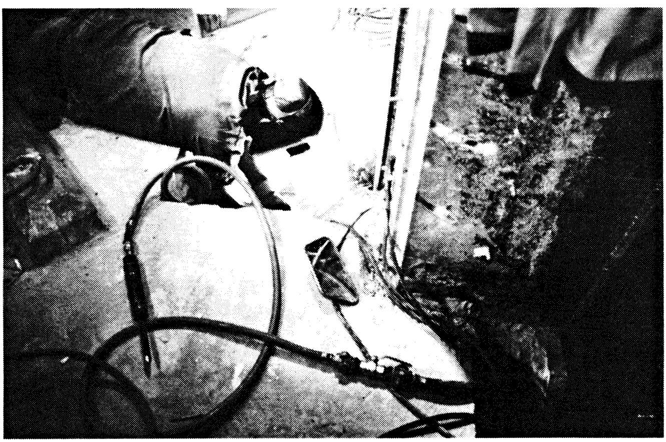

Fig. 10-2 - Welding of steel reinforcing plates to existing column.

CHAPTER 11
THE USE OF EXISTING CONNECTION ELEMENTS
BY

JOHN DOMBROWSKI
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One major question asked was, "What are we going to find behind the marble and what is the condition of that existing work?"

The project team decided that it would be both economical and timely if we reused as much of the existing connection system with the new cladding design as best we could.

The existing connection system is shown in Fig. 11-1. The original marble and its supporting stainless steel bent plate shelf angle were not going to be reused. The two remaining clip angles are the outer clip angle, which we call the B Clip and its connection angle back to the base building, which we call the A Clip were considered to be reusable. The B Clip had 2-inch horizontal oversized slots, and the A Clip had 2-inch vertical oversized slots. Both slots are ultra long as Jack will explain later. The A clip was bolted with steel-welded studs back to the building with Wiz-lock nuts. Basically, a Wiz-lock nut is a flanged nut with serrations on it. The serrations are intended to prevent the nut from backing off the stud. The bolts connecting the two clips together were mild steel and were not going to be reused.

The base steel was also examined for corrosion that may have developed over the years. It was surprising that only minor repairs were required to address corroded steel, and the repairs that we performed consisted of removing the corrosion and coating the steel with a Conlux 381 paint 8 to 10 mils thick.

The focus of examining the existing conditions was to find a design solution for the new cladding of which three elements were of major concern: the granite panel which includes the thickness in the kerf design, the shelf-angle design, and the supporting A and B Clips which were the elements of adjustment. We chose to keep A and B clips because there were over 176,000 of these angles on the building and therefore a very costly element to replace.

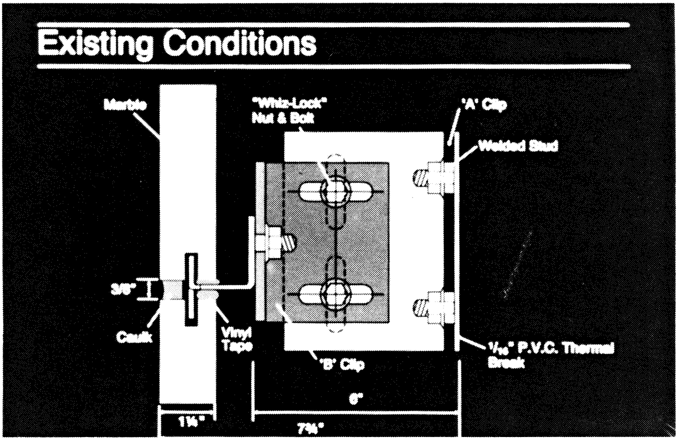

Fig. 11-1 - **Section through original marble panels showing existing conditions.**

CHAPTER 12
DESIGN AND TESTING OF GRANITE PANELS AND OF STAINLESS STEEL SHELF ANGLES
BY

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Design and Testing of Granite Panels

The new granite panels were designed based upon the strength of the selected granite determined by testing. The initial testing established that 1400 psi flexural strength could be provided by the Mount Airy granite. A quarry testing program was conducted to assure that only granite panels with this 1,400 psi minimum strength were fabricated for the project. This quarry testing program will be described by John Dombrowski in Chapter 13.

The panels were designed for a wind load determined by the wind-tunnel testing as 75 psf outward load and 45 psf inward load. The design of the new granite panels was performed using the factors of safety recommended by the National Building Granite Quarries Association (NBGQA). The factors of safety are three for granite in flexure and four for granite at the connections. This higher factor of safety at the connections is used because of the importance of connections and because of the greater degree of variability of connection strength.

NBGQA also establishes tolerances for the fabrication of stone, which was considered in the design. Tolerances allowed by NBGQA are quite liberal as shown in Fig. 12-1. The fabricated granite is allowed to be 1/4-inch thicker or thinner than specified, the location of kerf in a panel can vary by up to 1/16 inch, and the width of the kerf can vary by up to 1/16 inch. The tolerances are additive, hence, the thickness of the granite behind the kerf can have variations of up to 3/8 inch. In addition, the depth of the kerf can be 3/8 inch greater than what is specified.

We recognized that if we were to accept these standard tolerances that the design thicknesses of the granite panels would be much thicker. Therefore, we met with North Carolina Granite Company in Mount Airy to discuss tolerances. We were able to agree that the thickness of the panels would be held to plus or minus 1/8 inch from specified and that the depth of the kerf would be held to no more than 1/8 inch

deeper than specified. By controlling fabrication to these tighter tolerances, the design thickness of the granite was reduced by 1/2 inch, compared to what it would have been if standard tolerances were allowed.

Design phase testing of the granite included laboratory testing of full-length kerfs and ASTM C880 flexural strength tests on samples cut from the same slabs that were used to fabricate the full-length kerf tests. The ASTM C880 flexural strength tests were performed on granite panels as-received and after accelerated aging. In addition, short-length kerf strength tests were performed before and after accelerated aging to establish the loss of strength in the kerf as a result of exposure to the environment.

After granite panels for the building were designed based upon theoretical calculations, ten panels of this design were tested for both inward and outward loading. The average initial strength of the test panels under outward wind load was 590 psf. After the strength of the test panels was reduced for the wet condition and for loss of strength due to exposure to the environment, the panels still had a factor of safety greater than four for the 75 psf outward design load. Under inward load which stressed the thinner side of the kerf, the test panels had an average strength of 370 psf after the strength of the panels was reduced for the wet condition and for loss of strength due to exposure to the environment, which also results in a factor of safety greater than four for the 45 psf inward design load.

Testing was performed on some panels with sealant in their joints. This testing revealed that new cured sealant in the joints increased the strength of the joint by about 70 percent. However, this additional strength was not used in the design because sealant in the joint will deteriorate with time.

We also found that the connection at top of shelf angle is stronger than the connection at bottom of shelf angle by about 100 percent. This behavior will be explained later in the section on design of the shelf angle.

The flexural strength testing of the granite samples was performed in accordance with ASTM C880. The test samples were 2 inches thick, 3 inches wide, 22 inches long. The samples were tested in both wet and dry condition. The testing was performed on 300 samples obtained from the ten panels that were used for the full-length kerf tests. Two hundred of these samples were tested in the as-received condition and 100 samples were tested after 300 cycles of accelerated aging.

The ASTM C880 flexural strength of Mount Airy granite at the design thickness and at actual exterior finish was 1,620 psi in its stronger direction and 1,550 psi in its weaker direction. In the wet condition these strengths were 1,500 psi and 1,420 psi. These test results were consistent with the initial testing results.

As Ian Chin described in Chapter 3, an accelerated weathering test procedure had been developed by WJE during the investigation of the original marble panels. The testing of the original marble from the building determined that 300 cycles in the accelerated weathering test chamber is equivalent to about 25 to 30 years of exposure to the Chicago environment. This accelerated weathering testing was performed on both ASTM C880 granite samples and on short-length granite kerf tests specimens.

Fig. 12-2 shows a short-length kerf test on Mount Airy granite. It is a 3-inch length of kerf supported by a roller beneath the near end and by a short length of kerf at the far end. The sample was loaded by the test machine close to the kerf. Fig. 12-3 is a view of the kerf end of the sample after a full test load was applied. Samples such as these were tested before accelerated weathering and after accelerated weathering to measure loss of strength that will be expected in the kerf after many years of exposure to weathering.

ASTM C880 flexural strength of the granite was reduced by 12 percent due to exposure to 300 cycles of the accelerated weathering test. Kerf strength was reduced by about 18 percent. The granite at the kerf loses a greater amount of strength because it is thinner than the body of the stone and has a greater proportion of its area exposed to the temperature extremes. These strength losses were considered in the design of the stone.

The testing of the short lengths of kerf provided valuable information which was used to develop an accurate design equation for stone kerfs. Fig. 12-4 shows a comparison of the kerf strength analyzed by the equation and tested kerf strength. The agreement between analysis and testing is excellent.

There is a soffit section near the top of the building that overhangs the plaza. A decision was made to design this section in aluminum for simplicity of construction.

Design and Testing of Stainless Steel Shelf Angles

The design of the stainless steel shelf angle extrusions considered flexure, deflection of the angle, fatigue loading, and the effect of the shelf angle on the bolt that attaches it to the clips. The design was verified by load testing.

Contrary to popular thought, stainless steel can be extruded. Two fabricators who could extrude stainless steel shelf angles were located. The stainless steel extrusion process is similar to extruding aluminum but it is done at a very high temperature and a special die lubricant is used.

The extrusion of stainless steel begins with a billet of steel that is six inches in diameter, and about 18 inches long. The billet is heated to about 1,600 degrees F and is forced through the die at this high temperature, as shown in Fig. 12-5. Glass powder is used to lubricate the dies. The dies, shown in Fig. 12-6, wear out very rapidly so that many dies were used to extrude the stainless steel shelf angles required for the Amoco building.

After the hot billets were pushed through the die, the extruded stainless steel shelf angles were not straight due to the very high temperature used during the extrusion process. However, this was not a problem. The extrusions were straightened by first pressing them to reasonably straight and then they were pulled in what is called a stretcher-straightener. Tension load was pulled on the extrusion to force it into a straight condition and then after the tension was released, the extrusion remained straight.

Fig. 12-7 shows tolerances that were held after the fabrication process was completed. The extrusions were square and straight so that they would fit into the kerfs in the stone panels and so that they would provide uniform bearing to the stone. One of the most critical tolerances was that the shelf angle be straight within 0.063 inch so that the shelf angle would fit into a kerf that had a 1/16 inch undersize tolerance.

During the design of the shelf angles, consideration was given to fabricating a bulb at the ends of the upturned and downturned legs of the shelf angle to produce a stronger stone joint. The idea was that by having the bulb, the legs would bear deep in the groove to reduce the bending stress in the kerf and produce stronger connections. However, testing revealed that the bulb did not increase the strength of the

connection. As shown in Fig. 12-8, under outward load, the shelf angle rotates outward which causes the bottom leg of the shelf angle to bear against the outer edge of the stone below - the bulb is just pulled away from the stone. At the upper stone the rotation causes the end of the leg to bear against the stone deep within the kerf, whether there is a bulb or not. This behavior is consistent with the test results previously discussed in which the stone joint below the shelf angle is significantly weaker than the stone joint above the shelf angle.

Fig. 12-9 shows the final kerf design. The joints between the granite panels are 9/16 in. wide. The granite panels sit above a shim to adjust final height of the granite. Soft compressible tape is inserted between the bottom of the shelf angle and the top of the granite panels to create an expansion joint so that stacking will not occur on this tall building. The kerfs are filled with sealant to ensure uniform bearing between the shelf angle insert and the granite and to prevent water from filling the kerfs and then freezing. The stainless steel shelf angle is attached to a galvanized steel clip angle using a stainless steel bolt.

Since the design of the recladding used the existing clip angles, the clip angles required a high degree of adjustability to allow for differences between alignment of the original structural steel and alignment of the new granite panels. The clips were referred to as the A clip and the B clip. The A clip was attached directly to the building, the B clip bridged the space between the A clip and the shelf angle. The A clip has vertical slots to allow up/down adjustment; the B clip has horizontal slots to allow in/out adjustment.

The granite cladding is specified to be erected plumb. Steel industry tolerances for the structural frame of the building allow for deviation from plumb. Therefore, the clip angles must have slots or other means to allow for inward and outward adjustment.

The original marble cladding was previously installed on a column-by-column basis. For the recladding, it was decided that for aesthetic reasons, the horizontal joints at adjacent columns would align. Therefore, vertical slots were needed in the clip angles to allow for this adjustment.

The existing clip angles did have slots that are referred to as ultra-long slots because they are longer than the slot size defined by the AISC specification. The AISC specification does not disallow use

of the ultra-long slots, but it provides little guidance in the use of slots that are longer than standard size. To use the existing A and B clips with these ultra-long slots, the assembly was retrofitted to include covering the ultra-long slots with a plate washer and then tightening the A clip against the B clip using high-strength, load-indicating bolts. This design was performance tested in the laboratory.

Fig. 12-10 is a model of the connection with the clip angles, shelf angle, and stone panel. The building column is represented by the steel plate on the right, which obviously is much thinner than the building columns. The horizontal slots in the B clip are covered with the horizontal plate washers. Fig. 12-11 shows the opposite side of the A/B clip assembly. The vertical slots in the A clip are covered with the plate washers.

Two load-indicating bolts were used to attached the A clip to the B clip. The load-indicating bolt has a spline that projects out beyond the end of the bolt. A tightening tool holds the spline and turns it in a counterclockwise direction while the same tool turns the nut in a clockwise direction. When the torque that produces the desired tension is achieved, the spline twists and fractures off. In Fig. 12-11 the spline remains on the lower bolt and the spine has already been twisted off the upper bolt.

There is a vertical slot in the holes used to attach the A clip to the building. While these slots did not allow any vertical movement of the marble panels, the increased weight of the granite requires that a 3/8-in. diameter bolt be installed through the A clip into the building columns to prevent vertical movement. This bolt was threaded into a hole that was drilled and tapped into the column.

In addition to retrofitting the existing A and B clips, 12 special A and B clips were designed to allow for the additional adjustment that would be necessary to adjust clip angles to produces a plumb granite cladding.

The modified A and B clip designs were tested in the WJE laboratory. The purpose of the test program was to verify that the clips had adequate strength and to verify that the A clip would not slip against the B clip. This testing was performed by applying load in the laboratory while measuring movement of the A and B clips using sensitive electronic displacement gauges. Fig. 12-12 shows the test in progress. The shelf angle was pulled outward by hydraulic rams acting through six steel brackets to

simulate uniform outward load. The shelf angle was pulled down by load cells, to simulate the weight of the granite panel. There was no slip of the A clip relative to the B clip when the plate washers and bolts were properly installed. No fractures occurred, and the failure mode at very high load was by the clip angle and shelf angles deflecting into the plastic range. Fig. 12-13 is a photograph that was taken after a load equal to ten times design load was applied.

The bolt attaching the shelf angle to the B clip is stainless steel. Stainless steel was chosen to provide the greatest assurance against corrosion. The bolt was chosen to be a high-strength bolt so that it could be tightened to proof load. By tightening to proof load, variations in bolt tension due to pulsating wind loads would be eliminated.

Stainless steel bolts require special lubrication to allow them to be tightened to high tension. If the appropriate lubricant is not used, the threads will gall and cause a torsion failure in the bolt prior to reaching the required tension. The lubrication that was chosen is a dry film lubricant so that grease would not be present at the project to avoid workers staining the stone.

A review of corrosion aspects of the design which used stainless steel, galvanized steel, and carbon steel was done by a corrosion engineer to verify that dissimilar metal corrosion would not occur. The review concluded that corrosion would not be a problem, because stainless steel which is the more noble material has less mass than the carbon steel or galvanized steel. In considering corrosion in dissimilar metals, if the more-noble material has a smaller mass than the less-noble material, dissimilar metal corrosion will generally not occur.

The design was tested using full-size mockup testing. The testing was performed at two locations. Air infiltration, water penetration, and preliminary structural testing were performed at CRL in Florida. The final structural load testing was performed at the WJE laboratory in Northbrook, IL using a three-panel-high test fixture, shown in Fig. 12-14, that was designed to load the panels and their connections to four times design load. Fig. 12-14 shows assembly of one of the three stone panels onto the test frame. As a part of this test procedure Bill Weis, the stone installer, worked with WJE at the WJE laboratory to erect the first

test assembly. The test mockup was assembled using the same procedures and the same construction equipment that Bill will describe in Chapter 16.

Finally, after the test assembly was completed, the panels were loaded to four times design load, 350 psf. Deflections and stresses in the support systems were measured. Twelve tests consisting of three repeats of four different panel conditions were conducted. All of the assemblies supported the test load of four times design load with no failure and no distress.

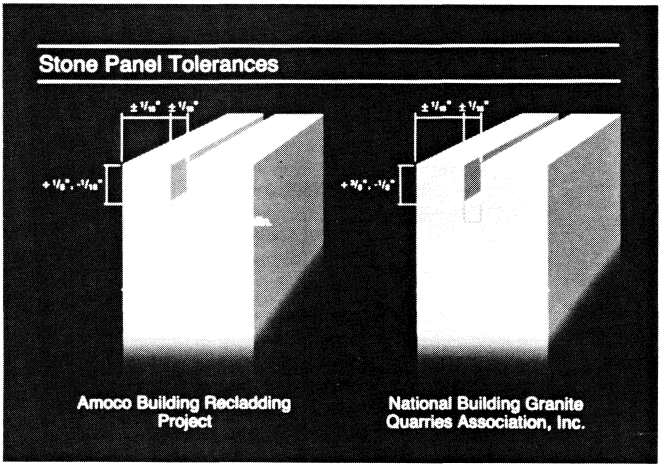

Fig. 12-1 - Stone panel tolerances.

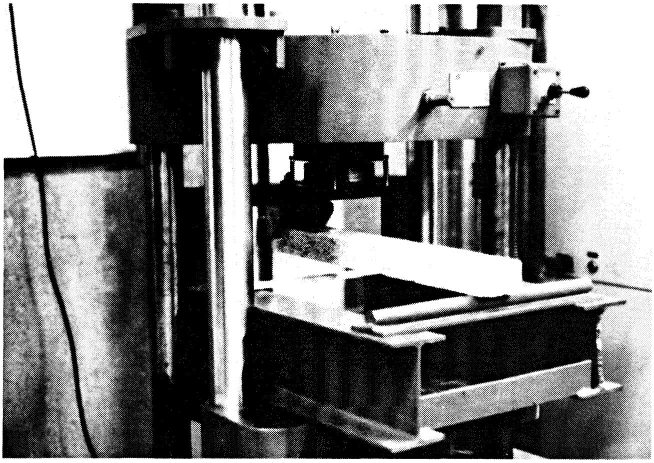

Fig. 12-2 - Short length kerf test of sample of granite.

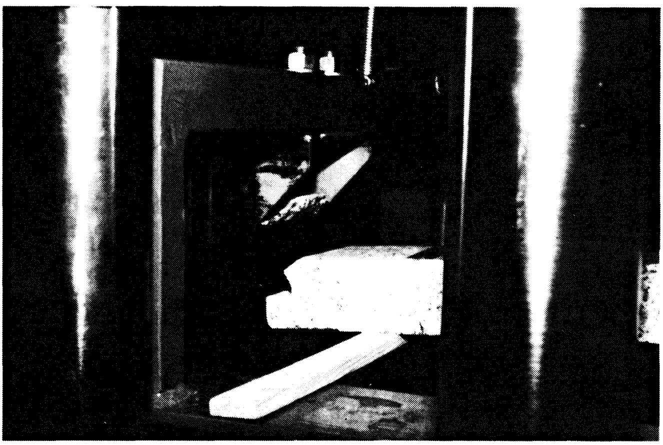

Fig. 12-3 - Fractured end of kerf sample after testing.

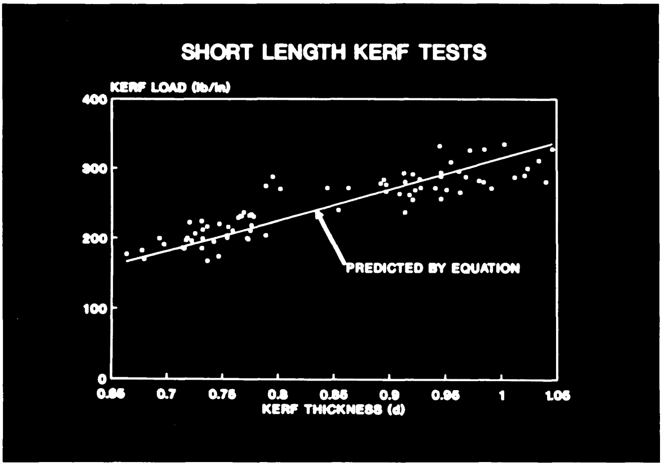

Fig. 12-4 - Comparison of kerf strength by analysis to tested strength.

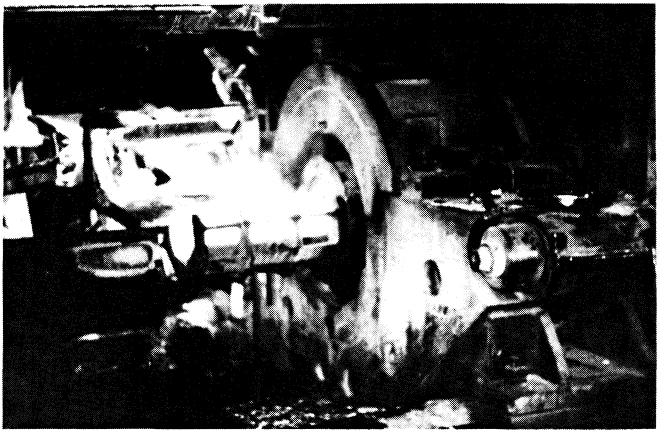

Fig. 12-5 - **Stainless steel billet at high temperature being pushed into die.**

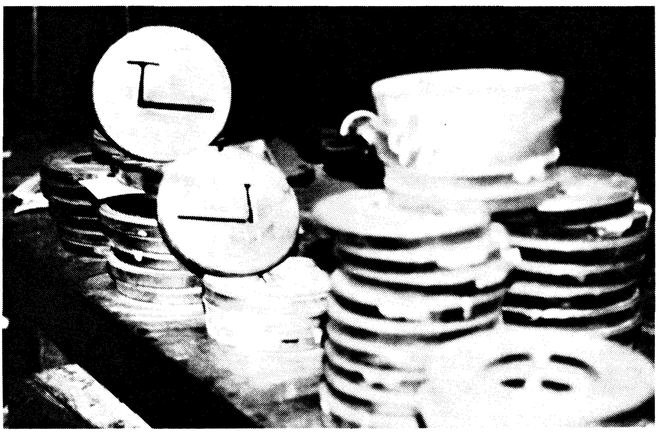

Fig. 12-6 - **Dies used to extrude the stainless steel shelf angles.**

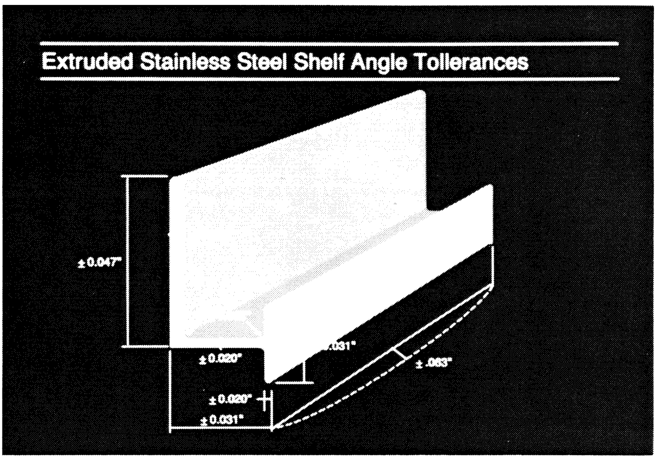

Fig. 12-7 - Tolerances used for stainless steel shelf angle.

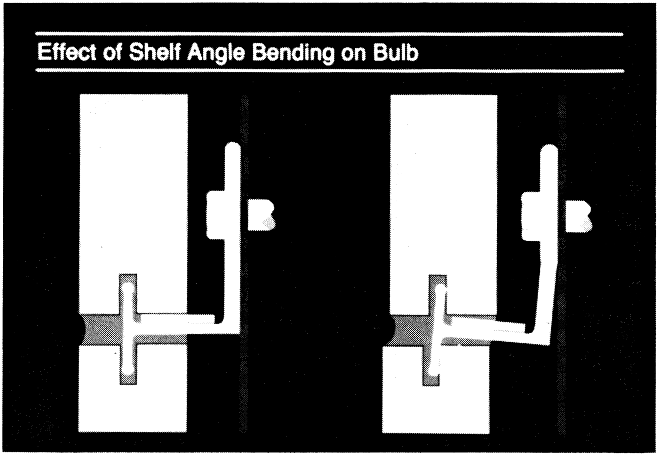

Fig. 12-8 - Effect of shelf angle bending on of bulb at end of insert.

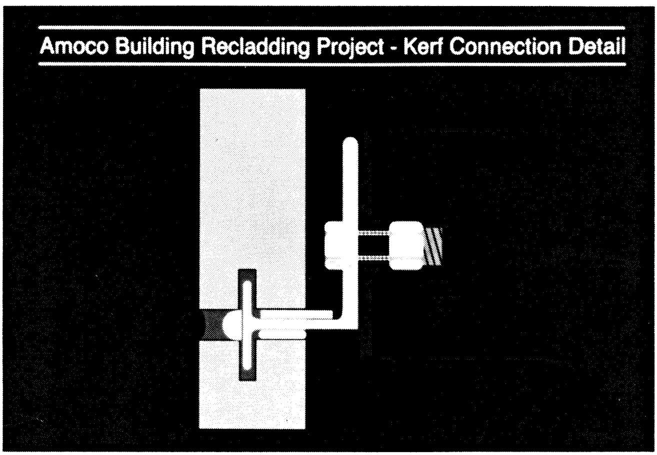

Fig. 12-9 - Final kerf connection detail used for recladding of Amoco building.

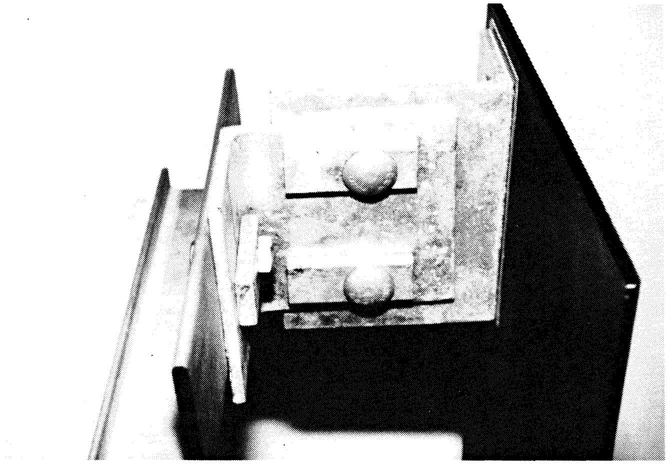

Fig. 12-10 - Model of clip angles, shelf angle and stone panel below.

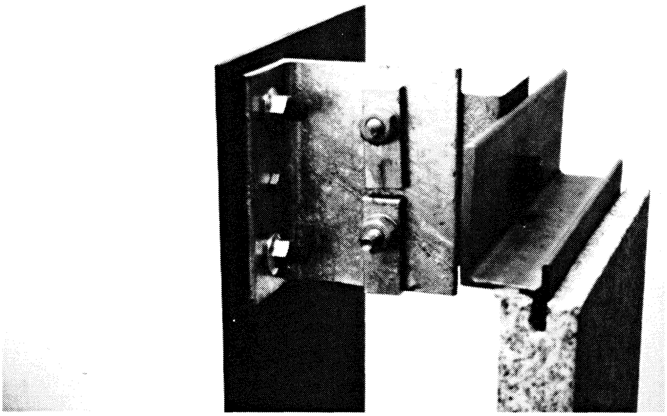

Fig. 12-11 - Opposite side showing covered vertical slots on A clip.

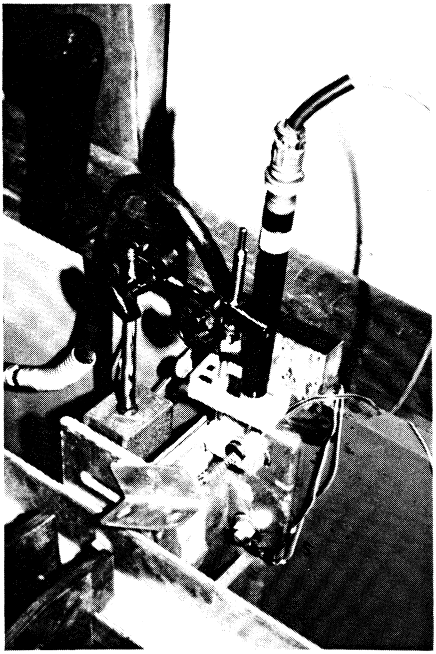

Fig. 12-12 - Test of steel connection system in WJE laboratory.

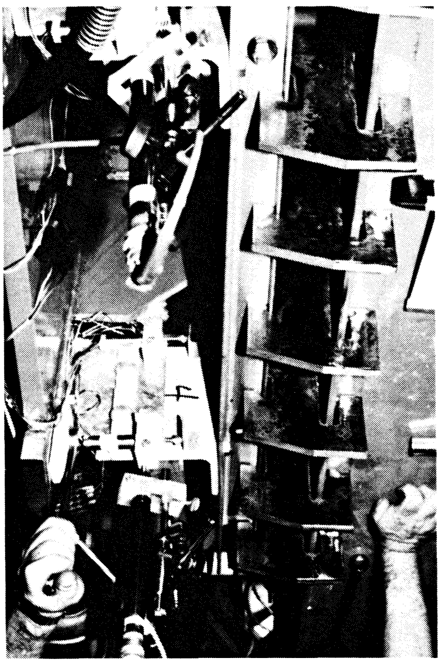

Fig. 12-13 - Bending of shelf angle at load of ten times design load.

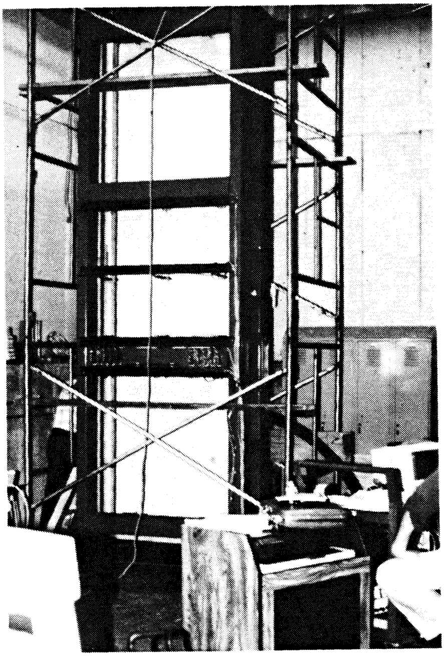

Fig. 12-14 - Structural load test set-up in WJE laboratory

CHAPTER 13
QUALITY CONTROL PROGRAM FOR QUARRY OPERATIONS AND PANEL FABRICATION
BY

John Dombrowsk
Project Manager, Amoco Properties, Inc.

All of the issues involving the granite panel design were based on testing that was performed in the laboratory on granite samples. The project team wanted assurance that the actual material that was going to be used for the cladding met our specifications for both strength and tolerances.

In order to achieve this requirement, a detailed quarry control program was developed for quarry operations and panel fabrication. But first an understanding of the quarry itself had to be made.

Since granite is an igneous rock formed by cooling, a knowledge of the natural characteristics of the quarry such as rifts was required. Rift is a directional fold from which the granite flowed and formed. The angle of the fold affects the strength properties of the granite. The greater the angle with respect to horizontal, the weaker the material. This condition was confirmed by testing samples cut parallel and perpendicular to the rift that Jack mentioned earlier.

Therefore, the first criteria for the quality control program was to identify areas of the quarry that have significant land massings and rift directions. By locating these features in the quarry, areas of the quarry that have acceptable material could be identified so that the blocks extracted could be cut as close to parallel to the rift as possible. No blocks were accepted in which the angle of the rift exceeded 20 degrees from horizontal.

Preliminary field point load tests and strain measurements were performed to establish a measure of quality and homogeneity of regions of the quarry. The number of tests would depend on the size and variation of the region. The regions of the quarry that produced point load test results below that of the equivalent 1,400 psi flexural strength were deemed unacceptable. Any blocks quarried from these regions were identified and were not used for the project.

Upon selection of the regions or areas of the quarry that has acceptable stone, a ledgeline was blasted away from the quarry. This ledgeline was approximately 50 to 100 ft in length. From there the

line was then divided into individual blocks. From each line, a block was randomly selected as a test block. The number of test blocks per ledgeline was dependent upon its size and, again, material homogeneity. Approximately 150 blocks were tested out of the 1,500 blocks quarried for the job.

At this point, a detailed program for qualifying the ledgeline's blocks was introduced. John Logan then performed point load tests on individual test blocks to determine if the blocks have adequate strength properties. If the point load values were at or above the equivalent 1,400 psi flexural strength, a test block was selected and slabbed parallel or near parallel to the rift. One slab of the test block was selected for testing. The test slab was cut into ten ASTM C880 test specimens, five in the vertical direction and five in the horizontal direction.

The ASTM C880 test specimens were tested in the dry condition. If the lowest average of a group of tests in the same direction was above 1,400 psi, all the blocks quarried from that ledgeline were released for fabrication. If the lowest average of a group was below 1,400 psi, then an additional slab in the test block was tested again following the same procedure. If the average results from both the first and second test slab specimens were below 1,400 psi, the block was then rejected. John Logan then reexamined other blocks that were removed from the same ledgeline and the procedure was repeated with another selected test block. If the results were below minimum specifications after this run of tests, the blocks from the entire ledgeline were rejected. All ledgelines, individual blocks, and test blocks were then identified on the quarry map for future reference.

Upon completion of the block testing, the material was then slabbed, finished, and dimensionally cut into the specified panels. The panels were then checked in respect to the dimensions, appearance, finish, and tolerances.

One major note is that all the testing that occurred in the quarry, could not have taken place without the cooperation of the North Carolina Granite Corporation. This was truly a team effort.

This concludes the design phase of this presentation. Bernie Simmons will now lead the team that will present information on the construction phase of the project.

CHAPTER 14
SITE SAFETY
BY
Bernie Simmons
Engineering Supervisor, Amoco Oil Company

Table 14-1 shows a listing of some of the quantities of materials we had to contend with while working up to 1,100 ft in the air. These obviously made all our plans very important.

I will begin by talking about the site constraints that were faced by the team as we began to plan for the stone replacement. There were times when the requirements and the constraints could have been in the way of expeditious construction, however, it was my responsibility as project engineer and then finally project manager to assure that they were not.

The first thing that we addressed was site safety for the pedestrians around the buildings as well as the tenants and employees in the building.

We wanted to insure that access to the construction site or the hard-hat area was limited while at the same time we wanted to make certain there was an easy and obvious building evacuation path from all exits in the event of an emergency.

Finally, we wanted to make it obvious to users of the building that we, the construction team, recognized that their building was still occupied and was their place of business.

Canopy design had begun in 1988, but once the decision was made to proceed immediately with the reclad, we proceeded to complete the design of a pedestrian canopy that would be inviting. The aesthetic qualities of the canopy were important because the building serves as our corporate headquarters as well as the offices of other prestigious businesses. Fig. 14-1 is a partial view of the canopy.

We used the recently completed northerly street entrance as the model for the appearance and carried that theme and color scheme all around the building on the canopy structure. Voy Madeyski, who was the designer for the original northerly street work, was the architect that we used for the aesthetics of the canopy. The structural design of the canopy was performed by Wiss, Janney, Elstner Associates, Inc. (WJE). We wanted to project an open or airy feeling as someone walked through the canopy. Other construction canopies that we had been in were cave-like. We were looking for openness.

We also wanted to provide a sense of security to pedestrians as they walked through at any time of day or night. To that end we installed over 700, 2 ft by 2 ft or 2 foot by 4 ft fluorescent light fixtures. We also installed 21 security cameras at strategic points in canopy. These cameras were visible to the pedestrians and were monitored at the central building security control console in the building. Additionally, the canopy was added to the normal daily rounds of the Amoco security force.

The design of the canopy was interesting due to the fact that the sidewalk around the building was essentially a bridge structure. The wind loading required that we use concrete ballasts around all of the posts. We used MDL plywood for the painted surfaces for its long life and paintability. We wanted no rain dripping in to annoy the users, therefore, we designed the canopy with a roofing membrane system with scuppers every so often to collect and drain water off the roofing.

There were provisions for access to the roof of the canopy for repair of the membrane and for sweeping up of construction debris. The roof of the canopy was visible from offices and we were very sensitive as to how we were viewed by the tenants and our own management. Where access to the roof was frequent, handrails were provided.

Not only did the canopy protect the general public from construction, it served as an effective barrier for keeping the public out of the construction site. Every major door was covered by a canopy. However, the logistics were such that not every door was accessible from the street. For example, the southwest door in the southwest corner of the building was in the middle of the material movement pathway.

All doors, however, were kept unlocked during business hours and there were signs posted on the jobsite which pointed in the direction of the closest emergency exit from the plaza for use in a building evacuation.

There were several convenience issues both for the building occupants as well as for the contractor that arose and that were complicated by the nature of the building. Erection of the canopy was one of them. The erection sequence of the canopy was planned in great detail to insure that all entrances to the building were protected from the overhead work going on, that no one had an overly long walk, that no neighboring

entrances were under construction at the same time, and that handicapped access was maintained at all times to the building. This meant that the canopy construction needed to be staged. We could not just start at one point and work our way around. You will note on the schedule that some work was done only at night. This was done to insure the continued access to the building during the day for the disabled. In Fig. 14-2 you can see the night work which we were working on.

You can tell also that the coordination for the construction of something as simple as a canopy required a great deal of time and thought by everyone. In fact, I am sure this is the only construction site in the city where the labor foreman received a letter of thank you from one of our disabled employees thanking him for his and his company's attentiveness to his needs.

Because of this staged approach, the canopy took about five months to complete. During this phase, and as Roger indicated throughout the project, we kept the building occupants informed about which entrances were to be closed and when. Not only did we issue general tenant and employee announcements, we also provided signage at the effected entrances in advance of the closures so there would be no surprises coming down an escalator.

Another major area of conflict between the convenience for the building occupants and the construction needs was the fact that the southeast door was the major entry point to the building which building management was unwilling to give up. Unfortunately, it also led right into the middle of an elevated material pathway.

In order to meet both the construction convenience needs as well as those of the building occupants, it was decided to allow closing of this exit for construction during the non-rush hour period between 9:00 a.m. and 11:30 a.m. and between 1:30 p.m. and 4:00 p.m. This required building and dismantling the materials conveyor twice every day, and to design it such that in a building emergency it could be dismantled quickly. Additionally, the supporting structure when the canopy was not there could not interfere with the entrance.

A worker was always posted at this location when the conveyer was in place in order to begin to dismantle it in an emergency. This solution was innovative and effective and shows what an open-minded relationship and a little give-and-take can accomplish.

Additionally, the logistics also demanded that access to the plaza be available from the street level. This was accomplished by building a raised-roof canopy section.

There were occasions when fork trucks and pickup trucks needed to back onto the plaza for loading and unloading of material. However, a fence in this area was provided and kept locked during off working hours as well during working hours when no one was using a passageway to keep the public out of the area.

Another requirement that the building management laid on us was to insure that the plaza pavers were not damaged. Obviously, this was before the plaza redevelopment project was approved. At that time, though, in order to insure ourselves that we would not have to replace the plaza on our appropriation, we covered all areas of the plaza with a double layer of scaffold-grade two by tens. As you, no doubt have noticed, the plaza looks remarkably new after removal of the planks.

Before Rick talks about the details of the logistics for the site, I want to close this portion with the guidelines that the building gave to us as we began the planning, and continually reminded us throughout the construction. I heard this statement way too many times: "This was simply an operating building with a little bit of maintenance going on. This was not a construction site."

It is now to Rick, Schal's general superintendent to explain to you how we accomplished that.

Construction

• 88,000	Marble & Granite Panels
• 88,000	Stainless Steel Straps
• 88,000	Stainless Steel Shelf Angles
• 170,000	Galvanized Steel Clip Angles
• 440,000	Plate Washers
• 720,000	Bolts

Table. 14-1 - Quantity of some of the materials involved in the recladding of the Amoco building.

Fig. 14-1 - Partial view of canopy.

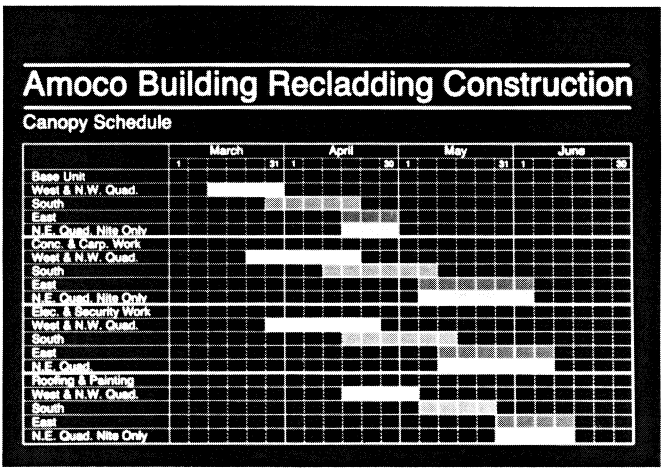

Fig. 14-2 - Canopy construction schedule.

CHAPTER 15
MATERIALS, HANDLING AND CONSTRUCTION LOGISTICS
BY
Rick Cantalupo
General Superintendent, Schal Bovis, Inc.

After hearing about the project from the other speakers, I am sure you can see how unique a project it was. We faced many unusual conditions. Besides being a super-tall building, it was also fully occupied. Throughout the project our number one priority was safety of the pedestrians, tenants, and of the tradesmen.

What we are going to show is that with the help of 18 different subcontractors and 150 workers, we accomplished the recladding of the fifth tallest building in the world and of tallest building that has ever been reclad.

What I will be talking about is our logistics plan and the preparation required prior to the start of the stone work.

We developed our logistics plan with a focus on the fact that the building was fully occupied. We first erected a canopy with appropriate graphics around the entire site, as shown in Fig. 15-1. Next we constructed a staging area on lower Randolph to handle the majority of deliveries and to minimize the truck traffic on the plaza level. That is indicated in the lower right hand corner of Fig. 15-2.

Material was brought from lower Randolph to the plaza by means of the material hoist. The material then went to one of the four vertical hoists which were placed in each of the re-entrant corners of the building. On the hoists, material went up to a landing platform and then on to a monorail system. It was then trolleyed to the installation point. At that point it was installed by workers who were on swing stages. All of the elevations had swing stages.

Now the removal of stone was handled the same way, only in the reverse.

The stages and the monorails were attached to needle beams which were secured at the roof level of the building, as shown in Fig. 15-3. This arrangement kept the weight of the stone panels directly off the stages during construction.

After a review of the construction loads that would be required on the plaza level, it was determined by WJE that it would be necessary to reinforce the plaza. The reinforcement was accomplished by a combination of installing new shear reinforcement in the beams, and by installing a grillage of steel beams and steel plates over the surface of the plaza area, as shown in Fig. 15-4.

Early on in the project, we reviewed our hoisting scheme and considered using two hoists in lieu of four, but we determined that this would lengthen the time for the stone erection and it was not as safe. Therefore, we decided to use four hoists in what we called the quadrant scheme.

The quadrant scheme allowed us to remove stone in two quadrants and install stone in the two remaining quadrants. Each quadrant had its own hoist and serviced half of two elevations. This half-of-two-elevation scheme gave us the shortest distance to the hoist. This is indicated in Fig. 15-5.

There was a significant amount of work necessary before we could start setting stone. Early on we decided to prepare the wall to receive the hoist. We needed to remove stone and expose the steel columns for the attachments of the hoist itself to the building. We did this by utilizing the house window-washing rig for access. This process eliminated the need to construct a temporary rig which would have slowed down that hoist installation.

One of our concerns was to keep water as well as welding sparks or slag from the hoist tie-in operation out of the wall cavity. This was accomplished by a metal flashing and caulking the opening.

The staging area on lower Randolph is shown in Fig. 15-6. We were allowed to do this after reviewing it with the City of Chicago and receiving a permit to close a portion of the street. It was an ideal location and helped reduce the congestion of vehicular traffic on the plaza level.

In order to get materials from the staging area to the plaza, we installed a material hoist. In order to do this, we had to penetrate Randolph Street which we prepared a series of structural drawings with calculations and submitted them to the city of Chicago for review. They issued a permit and we were allowed to remove the concrete section of the street while leaving the structural steel untouched in order to install the material hoist.

While most of the material-receiving activities took place on lower Randolph, we still required some storage on the plaza level for efficiency. A few material storage shanties were constructed at the plaza level with the same aesthetic concern as the sidewalk canopy itself.

While that work was going on in the project, the fabrication of the hoists was taking place at USA Hoist's yard. A mockup of the tower and the tie-ins to the actual configuration of the building was constructed to verify constructability. This mockup was very helpful and led to things such as painting the hoist white and not the standard red.

Again, concurrently while that work was going on, we were installing the monorail system. It was installed off a motorized stage. Monorails were placed at levels 17, 34, 52, 69 and the roof. At all the levels except for the roof, they were attached to the structure and then closed off with plywood.

Fig. 15-7 is a view of the monorail at the roof level and you can see the trolley that was attached to the aluminum monorail which then had a chain-fall attached to it. The chain-fall was used to raise and lower the stone panels at the point of installation.

Fig. 15-8 shows where the E-hoist penetrates the plaza level from lower Randolph and it shows the installation of the steel beam grillage to allow for the construction loads. This was on the plaza level.

In order to have all the hoists and the stages ready at the same time, we needed to get material to the roof before the hoists were operational. Since the material for the stages was too large to fit in the building elevators, we brought it to the roof by use of a helicopter.

The needle beams used to support the swing stages and monorail were attached to the existing window-washing machine track which circled the roof.

Prior to attaching any of the needle beams to the track, all of the pedestal supports for that window-washing track were checked for their integrity and all the welds were checked.

In Fig. 15-9 you see the needle beams attached to the window-washing track at the top and they are ready to drop cables for the swing stages.

In Fig. 15-10 you see the actual hanging of the custom-configured stages in preparation for stone installation. They were configured to fit the chevron shape. They also were custom in the sense that they

had walk-through stirrups which gave easy access to the men, and the stone which was hung off the monorail system by cable and chain-fall could then move easily in the trolley method in front of those stirrups.

Again, while the swing stage work was going on, we were beginning to install our hoists. Once again because of plaza loading conditions, we decided to use pendant booms attached to the building to minimize the weight on the plaza. The equipment plus the hoist plus the cranes was too heavy for the plaza so we installed these booms.

Unlike typical conditions where you excavate a pit, pour a concrete pad, and place your hoist on it, this project had five levels below the plaza. Of course, these levels were occupied by atypical tenants: a bank, computer room, restaurant, and even a densely-piped boiler room. This prevented us from shoring the plaza structure below the hoists. The solution was to cantilever and hang a grillage support system for the hoist from the structural columns at the corner of the building.

Once the grillage was in place, we began installing the machine room components. We followed with the reinforced base sections of the tower and the cab itself.

Fig. 15-11 shows how the hoist functioned during the erection mode. At this point the hoist was placed in running operation, but for the erection crew only. They would take the running car to the highest point, to the highest limit set on the hoist. They would then exit from the cab through a hatch at the top and climb by ladder to a temporary upper work platform. That would be exiting the cab, climbing up a ladder which was stationed here, and they would climb to this upper work platform.

From the upper work platform they would perform the tie-in work that attached the tower to the building itself. It was the only way to really access up close to the building. They would then climb from the upper work platform to the self-climbing cathead and stack new tower sections.

The new tower sections were installed at the rate of 38 feet a day. This procedure minimized the amount of climbing for the iron workers. As the tower got up higher--20 or 30 or 40 stories--obviously they could never climb that height so they would take a running car as I said, climb out of the car and then access through the ladder.

This is another day at the office. Fig. 15-12 shows the actual stacking of the tower panels themselves. They were raised to the top of the tower by means of that boom that was atop the self-climbing cathead. These panels were attached to a skinner line at the base of the building to prevent the panel from spinning or striking the building during its ascent to the top. Once all four panels were secured, the hydraulic self-climbing cathead would move up on the newly erected panels and repeat the process.

One problem that we faced early in the development of the hoist was "how do you get from a hoist 1,100 feet in the air to a swing stage?" This was accomplished with a landing platform. In Fig. 15-13 you can see the relationship of the hoist, the landing platform, and the swing stages. The landing platform, of course, was behind the hoist in this instance.

Not to interfere with the installation of the stone during the day and using weekends for any lost days that we had for makeup, we would jump the landing platform during the evening. Of course, we had to illuminate it heavily and we never jumped more than one tower each night.

The way that the jumping was performed was that the landing platform which was bolted to the tower was unbolted and then it was pulled up by using these electric motors which were located at the top of the building.

Bill Weis from W. R. Weis was responsible for the stone removal and replacement. He is going to show you how he did it.

Fig. 15-1 - View of sidewalk canopy around perimeter of building site.

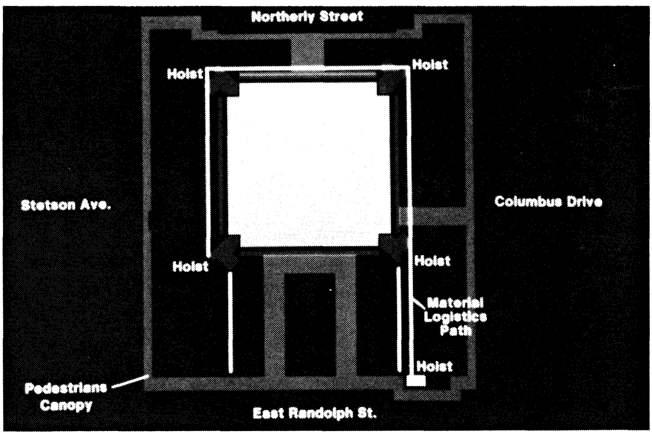

Fig. 15-2 - Site logistics plan.

Fig. 15-3 - **View of needle beams used to support stages and monorail.**

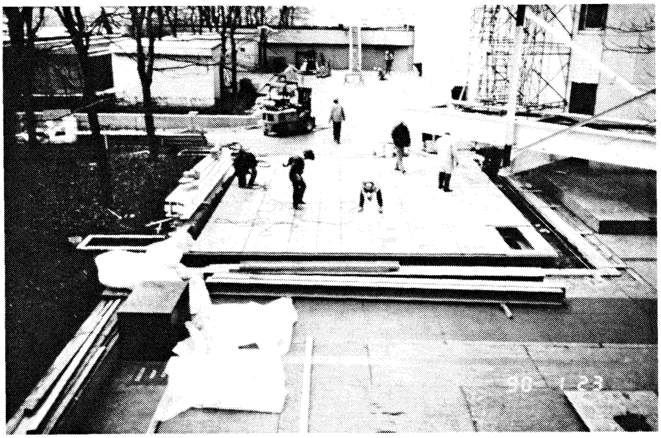

Fig. 15-4 - **Grillage constructed over plaza areas to support construction loads.**

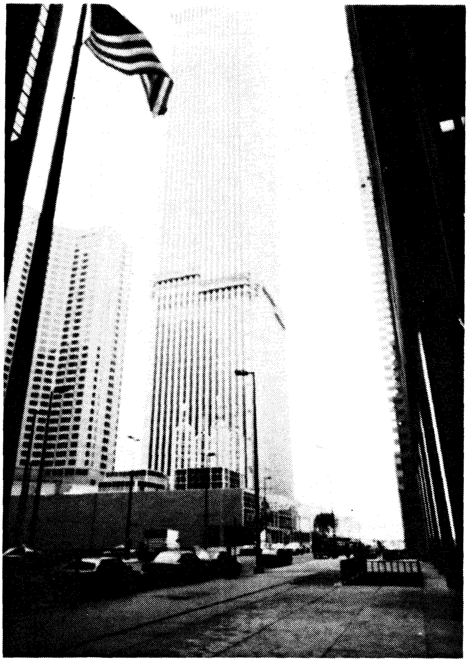

Fig. 15-5 - **View of hoist and stages in each quadrant.**

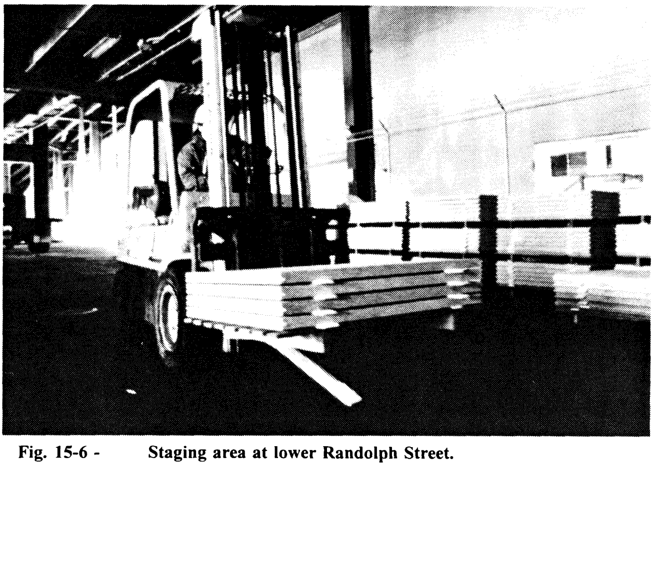

Fig. 15-6 - **Staging area at lower Randolph Street.**

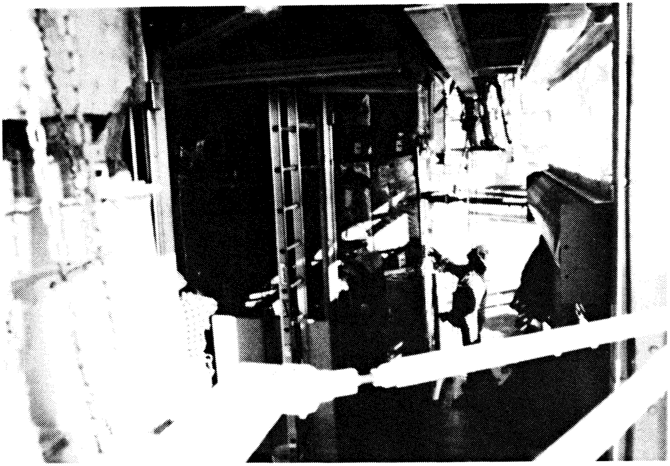

Fig. 15-7 - **View of monorail, trolley, and chain-fall.**

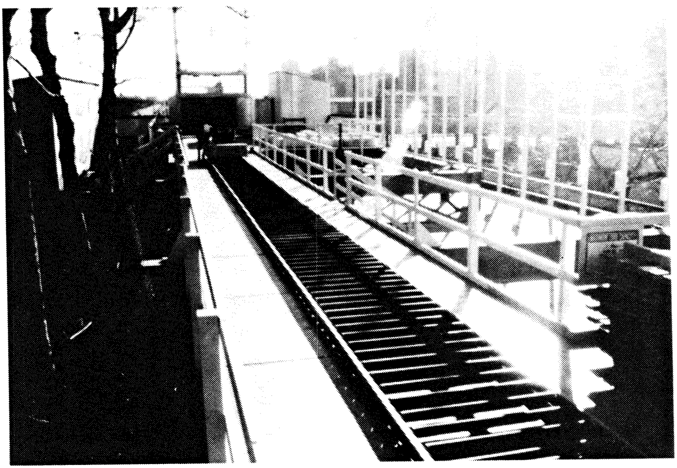

Fig. 15-8 - **View of E-hoist from lower Randolph and conveyor used to move panels.**

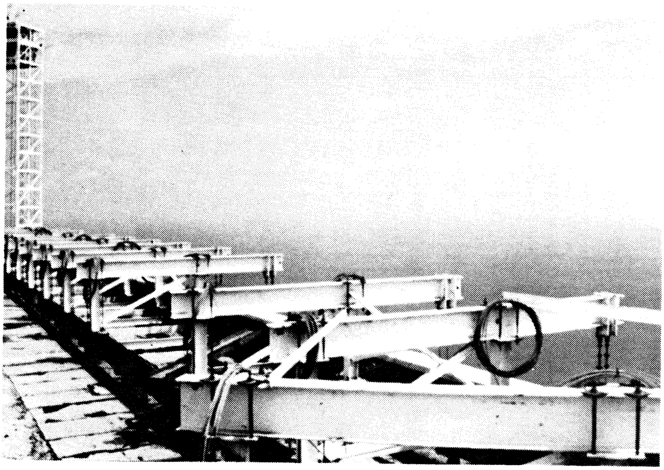

Fig. 15-9 - View of needle beams attached to window washing track.

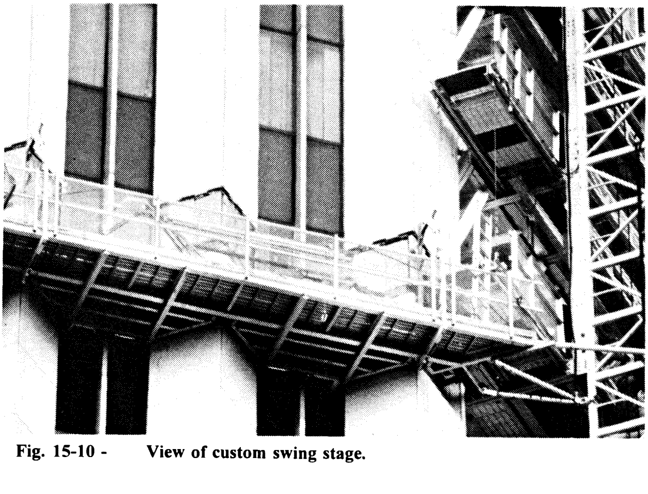

Fig. 15-10 - View of custom swing stage.

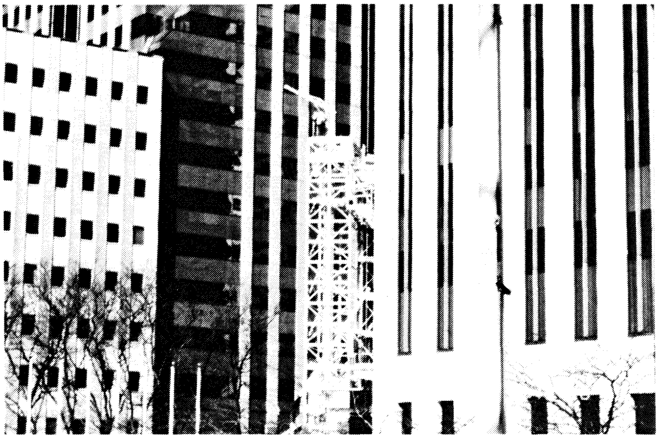

Fig. 15-11. View of hoist under erection.

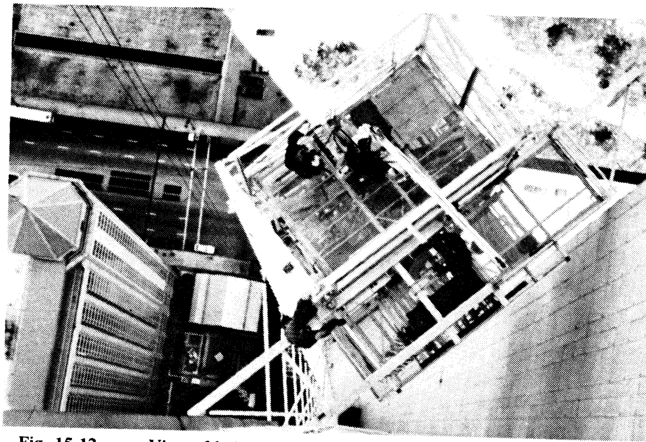

Fig. 15-12 - View of hoist under erection.

Fig. 15-13 - View of hoist landing platform and swing stages.

CHAPTER 16
REMOVAL OF ORIGINAL MARBLE PANELS AND INSTALLATION OF NEW GRANITE PANELS
BY

William R. Weis, President
W. R. Weis Company

I was in charge of all phases of marble removal and granite installation for the recladding project. I have broken it down into four areas.

Material delivery.

Delivery of granite panels from Mount Airy, North Carolina started approximately ten months prior to installation to build up a stock to insure a constant flow of material to meet the project's schedule. Granite was shipped to the warehouse vertically in bundles of 18 pieces, approximately 8000 pounds per bundle, with 6 bundles on each trailer load. This was the quarry's standard way of shipping. Shipping damage throughout the project was minimal.

At the warehouse the bundles were opened, each stone was inspected, and then placed horizontally on skids, six (6) pieces to a skid, approximately 2,700 pounds and numbered for installation purposes. They were placed horizontally not only for trucking to, but also for handling at the site. It was much safer handling 6 stones in a horizontal position for movement around the site and for loading onto the hoist.

As shown in Fig. 16-1, we had quite a bit of material on hand.

Trucking to the site was by 18-foot-long flatbeds, six (6) skids per load. Flatbeds were used in lieu of a tractor-trailer for convenience in negotiating city streets throughout the day. Loads were either off-loaded curbside for the north and west elevations, or brought to the lower Randolph Street staging area for transportation on the E-hoist servicing the south and east elevations.

On-site handling

Removal and installation of stone were accomplished using a monorail system. Prior to the start of the project itself, a monorail system which circled the building at five levels, as previously described by Rick was installed at the 17th, 34th, 52nd, 69th, and roof, as shown in Fig. 16-2. Installation of the monorail took approximately four months and was accomplished using two powered, 30-foot stages.

The stages were suspended from an outrigger beam system attached to the building window-washing track. Scaffold suspension cable guides shown in Fig. 16-3 were incorporated on the outside of the monorail brackets and custom-designed spring-loaded tie-down brackets were mounted at the base of the building to keep tension on the cables.

The scaffold was secured to the face of the building with custom-designed tie-in plates or slides attached to the building window-washing track, as shown in Fig. 16-4. This track was located at both sides of every window and ran vertically the entire height of the building. The tie-in plates would slide in the track allowing vertical movement, but not lateral.

The monorail and cable guide assemblies were attached to the building with steel brackets fabricated to conform to the chevron shape of the structural steel behind the stone. When the existing marble was removed at each bracket location, the structural steel was then drilled and tapped for bracket connection bolts. The brackets were installed and then plywood custom painted to match existing marble, as shown in Fig. 16-5 was then installed and sealed to weather-protect the cavity.

After installation of all brackets within the area serviced by the two 30-foot scaffolds was completed, the aluminum monorail beams were mounted. This procedure was repeated all the way around the building, always starting at the top brackets because once the monorail beams were in place, you would have to disconnect the scaffold from the tie-in plates for moving past a monorail.

There was a custom-designed, 3-foot-wide, 4-point suspended aluminum scaffold system fabricated for the project. We went to the 4-point suspension to eliminate the use of safety lines. There were four 30-foot and two 20-foot stages on each side of the building that were suspended from the outrigger system.

The stages were fabricated in these lengths not only for structural reasons, but also to enable us to divide the building into the quadrants previously mentioned by Rick by splitting each elevation in half.

Manual pumps were used in lieu of electric motors for vertical movement of the stages for the following reasons:

1. Due to the amount and length of power cables that would have been required and the fact that the cables would constantly be in the way.

2. With the amount of motors required, 48 units to be exact, you could be certain that every day one or more motors would not be running for whatever reason.

3. Our vertical scaffold movement at any time would not be more than seven courses of stone.

Ladders were mounted at both ends of each scaffold for access from one scaffold to another. The work platforms were designed to fit the configuration of the building re-entrant corner. Each work platform consisted of three working levels, each level with its own monorail system, and then the roof.

All stone, whether marble or granite, was transported to or from the work platforms hanging vertically on a stone clamp hooked to a manual chain-fall that was suspended from the monorail, as shown in Fig. 16-6.

Damaged marble was surrounded by a nylon harness cinched tightly at the top and attached to the stone clamp with a screw-pin shackle. The marble would end up in a horizontal position while the granite started out in the same position for transportation on the hoist to the landing platform at the base of the hoist tower. All stone handling from or to the work platform was accomplished using manual chain-falls and pallet jacks. These pallet jacks were on the lower platform and also on the working platforms themselves.

At the hoist platform, the stone was raised or lowered to the ground using electric chain-falls. Stone was transported to and from the corners using a forklift and conveyor system. There had to be close coordination between the handling of the old marble and the new granite because both had to use the same conveyor system but were going in different directions.

Work sequence

Work started at the third floor. Eight courses of marble were removed. The first course starting at the top was removed by saw-cutting through the stone using a diamond blade just below the top stone anchor, as shown in Fig. 16-7. Then by using a small prybar, we leaned the stone out just enough to attach a stone clamp onto the piece, as shown in Fig. 16-8. The stone clamps were then placed on all individual stones in the remaining seven courses before disconnecting their tie-in anchors as we worked our way down.

After the first course of marble was removed at the start of each jump, a stone alignment template, as shown in Fig. 16-9 (each chevron had one) which forms the shape of the chevron was attached to the window-washing track. Being unable to use control lines for installing the new granite, we had to devise a new method for its installation. Since the window-washing track was located at the jambs of every window and the existing marble fit between the tracks, it made sense to use the tracks as a guide. The template was fabricated to fit in the track as well as accommodate the additional 3/4 inch outward plane of the new granite. Segmented straight edges were then hung on the templates and tethered to the static line mounted above the templates for safety.

The existing A clip previously mentioned was then drilled through using a mag drill with a special 2-step drill bit attached to the B clip, as shown in Fig. 16-10. This enabled us to drill and tap the structure to receive a 3/8-in. bolt in the center of the two, 1/2-in. studs without removing the A clip. The mag drill was equipped with a quick-change chuck for speed and to ensure proper alignment for drilling and tapping. After all courses of marble had been removed and all A clips had received the new 3/8-in. bolts, we then started our granite installation.

The segmented straight edges previously hung were then put to use.

By bridging the plane of the existing marble above and the new granite below, the B clips were then hung. They were aligned and fastened to the A clips with 1/2-in. fasteners. Electric torque wrenches were then used to tighten the fasteners. The B clips controlled stone installation both for alignment and elevation.

The new granite was moved to its location using the monorail and was then installed using extruded stainless steel angles with stainless steel nuts and bolts fastened to the B clips, as shown in Fig. 16-11 and 16-12.

Once a complete course of new granite was installed, a segment of the straight edge which was the height of the stone just installed was removed and the procedure was repeated until we had installed seven courses of granite. Now remember that we had removed eight courses of marble but are only installing seven courses of granite so as not to have a closure course of stone.

The seven courses consisted of two floors of stone which was the area serviced by the work platforms. When this area which was designated as a jump was completed, we had removed 294 pieces of marble and installed 294 pieces of new granite in a five-day period. Also included in this piece count was the stone located in the corner or directly in front of the work platforms.

So as not to have a bottleneck on the work platforms, all stone removal and installation was performed first on the straight walls and then the corners. Corner work was handled using the same procedures.

The building was divided into quadrants with a hoist and work platforms servicing one-half of an elevation each way off the corner.

A jump consisted of the hoisting contractor moving the work platform up two floors on the night shift. As you can see in Fig. 16-13, the platform has already been jumped. We would come in early the next morning and jump all stages in that quadrant up seven courses of marble. This would always leave a one-course opening between marble and granite, stating again, eliminating having that closure course on every jump because remember the first time we removed eight but from now on we only remove seven.

The completion of the quadrants was staggered so as to do the same for the jumping of the work platforms in the evening. Staggered completions were kept possible using some overtime if a quadrant fell behind due to weather.

Before the work platforms were jumped, the seventh course of granite was checked for elevation not only within the quadrant, but also between quadrants using a level. Any deviation from level was then corrected on the first few courses of the next jump so as to maintain proper elevations.

We also had on hand a full course of undersized and oversized stone to compensate for previous errors in the marble setting. One oversized course had to be installed.

We caulked the building, as shown in Fig. 16-14 using the same scaffolds and basically the same procedures, only in reverse.

Crew Motivation

Money is the greatest motivator. We worked five, 9-hour days per week. This gave everyone almost six days of pay per week but had very little impact on their family hours. All additional overtime was split up between those who wanted to work. Knowing the guys as long as I did, I instilled a crew competition between the quadrants.

We also had what we called a "monorail party." The crew had to have something to look forward to due to the size of the project. It was almost like doing four, 20-story buildings but they could always see the monorail over their heads and it was something to look forward to. We did have five parties.

I would like to add at this time my personal satisfaction with always staying within our preconceived game-plan and for running and completing a project of this magnitude. It was one of a kind.

Fig. 16-1 - **Partial view of granite panels in warehouse.**

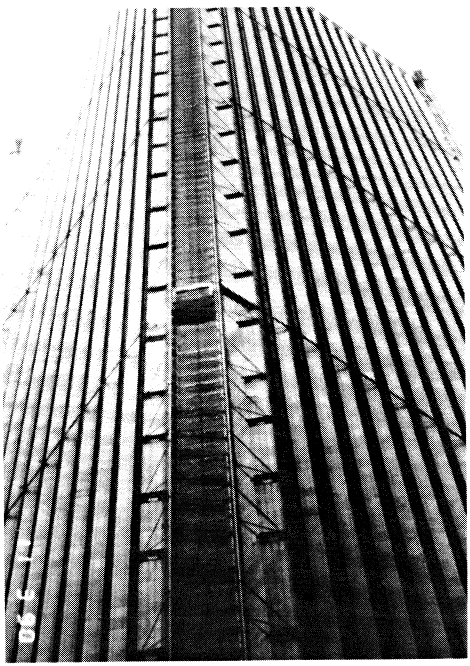

Fig. 16-2 - **View of monorail system on building**

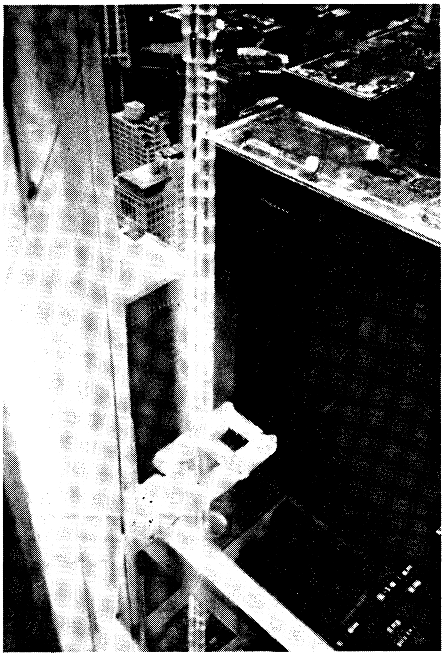

Fig. 16-3 View of scaffold cable guide. Ice formed on the cables and on the guilde on some winter days.

Fig. 16-4 - View of scaffold tie-in plates attached to existing window washer track.

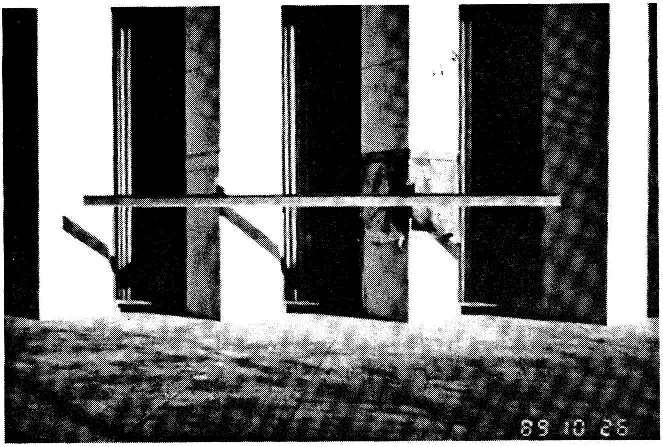

Fig. 16-5 - View of cover over monorail bracket.

Fig. 16-6 - View of marble panels being moved to the joist by the chain-fail suspended from the monorail.

Fig. 16-7 - View of sawcutting of panels prior to removal.

Fig. 16-8 - View of prying of panels for attachment to stone clamp.

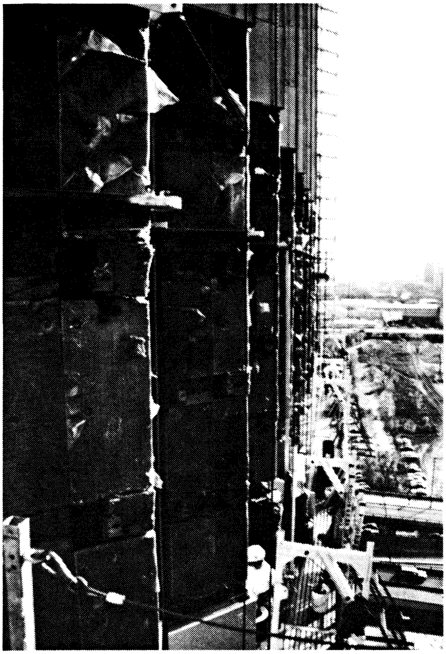

Fig. 16-9 - View of horizontal stone alignment template and vertical straight edge.

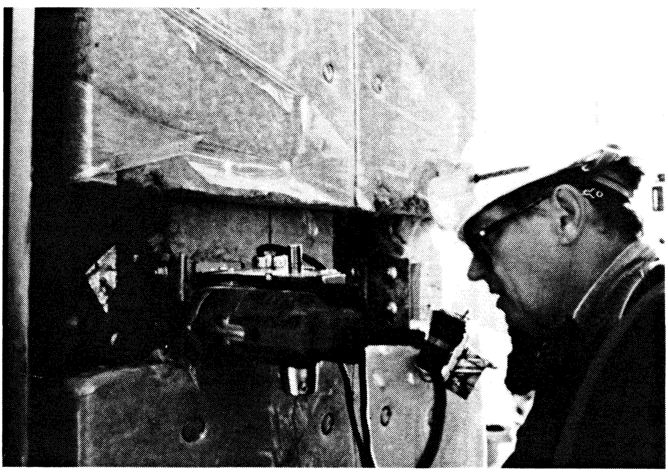

Fig. 16-10 - View of drilling of A-clip with a mag drill.

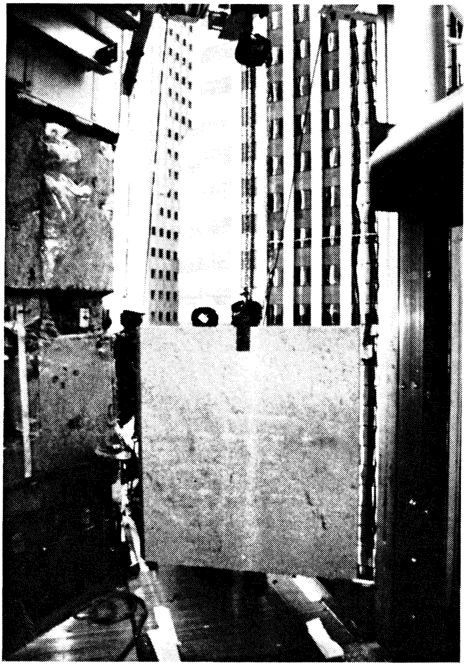

Fig. 16-11 - View of a new granite panel being moved to its position by the monorail.

Fig. 16-12 - View of installation of new granite panel and stainless steel angle.

Fig. 16-13 - View of work platform that was recently "jumped".

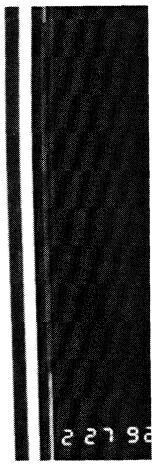

Fig. 16-14 - View of sealant installation.

CHAPTER 17
QUALITY CONTROL
BY
Bernie Simmons
Engineering Supervisor, Amoco Oil Company

Now that you know how we accomplished this work in the field, I would like to talk briefly as to how Amoco insured that it was done in the proper manner.

We viewed the quality control aspect of this project as a team effort, which required all of the participants working together. The workers, the craft supervision, the contractor, the consultants, and the Amoco project team all worked together to solve the problems and answer the questions, even to the extent that management of the Quality Control team was represented on the stages to understand the contractor's concerns, the problems associated with their existing conditions, or to emphasize our instructions.

The first step in creating a quality project is to insure that the design is buildable. Everyone participated in the drawing reviews and that paid off handsomely. It was the contractor who suggested that the B clips needed more adjustment than was provided by ordinary AISC slots. So we tested, as Jack mentioned, and approved long slots, and also used the multiple variations of the B and the A clips on the job.

We also wanted to insure that the building maintenance personnel would be happy with the job at completion. To that end we used various Amoco staff as inspectors. We selected six Amoco personnel that would be involved in the ongoing maintenance of the building to do the day-to-day quality control of the project. They were on the stages all day long with the workers. They answered questions before they became problems. They were empowered to make decisions regarding acceptability of the construction and we generally backed them up.

In order to make them effective, however, we had to develop quality control guidelines and train them. We put together a QC manual for the project. The manual was very detailed, nearly 60 pages long and contained a one-page checklist that they could carry with them, a narrative description of the work process that Bill has already described, definitions of terms, sketches of correctly installed components

showing the allowable tolerances which Jack went into, and pictures showing acceptable and unacceptable construction as well as some of the corrective actions they could take right there on the stages to get the job done right.

The inspectors were assigned to us before actual production stone setting began. Each inspector spent 16 hours in the classroom setting going over the manual and undergoing training that we designed. They were then able to spend time on their own reviewing drawings, visiting with the workers, and viewing the installation of the mockup.

Since the caulking was another operation--it was critical to the integrity of the facade--we sent these same inspectors to the sealant manufacturer's facility to undergo training and to get a handle on what made the sealant seal. I think you can all agree that they accomplished their task admirably.

Let's shift gears just a little bit now. From the beginning of the project, the topic most talked about in meetings was safety: safety of the worker, of the pedestrian, and of the tenants of the building. In fact, every meeting began with a safety discussion pertinent to the meeting as we required.

What characterized most of our safety plans was that they were proactive. We prepared an internal incident handling procedure with notification procedures and telephone numbers. This procedure was put together before any construction was begun on the site. This was to be used to advise Amoco senior management of any major accident or incident. I think that you probably get the picture that they were really concerned about this job. Fortunately, we never had to implement this call-down. We requested the same sort of call-down listing of our contractors and sub-contractors as well.

We also invited the fire department to visit and advise us as to their requirement for access for emergency personnel to evacuate injured workers in the unlikely happenstance that would occur. We made modifications to our logistics plan or set standing procedures for this access. We purchased and located stretchers and first-aid cabinets at strategic locations throughout the job site. As you can imagine with all the wood planking, fire was a major concern. We located dozens of fire extinguishers on the jobsite and set up a plan for checking the charging of them. In fact, as an example of the cooperation between the

building and the project, the building took on the responsibility of checking and then recharging these fire extinguishers on a regular basis.

We made available the CPR and first-aid training sessions for the contractors. Each major contractor that was routinely on site was offered a couple of seats to train supervision or journeyman-level workers in first aid and CPR. We felt that this would again emphasize our wish to be proactive right down to the worker level. This was done during working hours and the workers were paid for this time.

We knew one of the biggest concerns that the workers would have was the inability to get off the elevator should it break down enroute to the working platforms. We purchased from USA Hoist a custom-designed and custom-built emergency evacuation rig that could climb the hoist structure and retrieve workers trapped in the car. It was powered by a propane cylinder so they were not at the mercy of a power loss. We tested it, located it on site undercover, kept the cylinders charged, and never again blew the dust off it. We felt, however, that having it around was part of our contingency and prevention planning so that we could be prepared for anything.

We required Schal to provide a full-time safety person. We wanted to emphasize the importance we were placing on safety both to the contractors and to the crafts. His duties were to place signage, run safety meetings, perform safety audits on construction, insure that contractor-safety promises were kept, think about the work process and look for possible missteps. He got to be a real pain in the rear. And I guess that is the mark of a good safety person.

Amoco was very picky about housekeeping. This site was our world headquarters. As I mentioned earlier, there was only minor maintenance going on. We worked hard to insure that the site was clean and looking sharp.

Fall protection during construction in a building of this height was obviously very critical. During the construction of the hoist, the iron workers were exposed daily to this threat. We required and got compliance on the use of retractable safety lines for their unprotected work.

All appropriate signage was placed on the jobsite, not only those required by OSHA but also as reminders for the workers. Some of the signage was prepared to keep people and loads or materials off

certain plaza areas, and some signage was painted on the plaza to indicate allowable loading conditions. We were concerned once again about the plaza loading capabilities.

We developed an electrical lock-out procedure. We had to install the switch gear for the hoist on the fifth and lowest level of the working garage some distance away--five floors--from the machinery. With all the routine maintenance planned for the machinery, we needed to provide the electrician or even the maintenance worker with adequate protection.

Handrails, stairways, and ladder safety are another big factor in injury. We had a lot of stairs and handrails and we were adamant about their use and their correct construction.

Material drops was another issue that came under close scrutiny. We required tethered tools to insure that the many wrenches and hammers would not end up on the plaza. Fig. 17-1 was not staged; we have many such pictures.

Also shown in Fig. 17-1 are the plywood window protectors that we installed in the stages to protect the windows from nicks or chips or misguided stone panels. The other benefit to this was that it gave the worker on the stage some privacy from the office building and vice versa.

Of the same ilk was the addition of a flapper piece that bridged the gap between the stages and the building. It could be folded back into the stone. This was to prevent any of the little bits and pieces of material from dropping. You will note also that the stages were fabricated from checker plate and not grating again, to keep all the material in one place.

We were constantly performing preventive maintenance on the hoist machinery as I mentioned earlier. Every week, one hoist underwent some sort of maintenance procedure. Most of the time this was done on weekends, early mornings, or late afternoons so as to minimize the impact on the schedule.

In addition to the previous administrative or contingency planning safety items, there were several other major issues that were important to the execution of the work.

The weather played a major role. We were constantly monitoring the weather radio for storms. We were concerned not only about rain or snow, but also high winds. The City of Chicago Building Code requires that hoists shut down when the wind exceeds 35 miles per hour. To monitor this, we had wind

gauges placed on the hoist towers at the half-way points and also took readings on the roof and at grade. We shut an individual hoist down based on the readings in each corner. As you can see, the other wind issue that we contended with was the wind whipping the cables. The reason for the cable guide, I think, is self-evident in Fig. 17-2. These guides can take credit for eliminating damage to stone, both marble and granite. We had plenty of high winds, over a hundred cables that spanned the height of the building right next to the building, but never damaged any stone.

Other environmental factors effected our work. Rain, snow, and ice kept the workers off the elevated platforms because of slippery conditions. Additionally, there were several days that dawned beautifully blue and perfect winter working conditions, but the elevators were not going. There was an ice coating of up to 1-1/2-inch thick on the cables at the top. We, of course, waited for the sun to melt the ice.

I think you can see that safe working conditions were a high priority to us. The results of the project were good. Our lost-time accident rate was well below the industry average for building construction. Additionally, due to our preventive measures, we did not drop a panel.

Finally, as you no doubt have gathered by now, this was not a typical project by any meaning of the word. As regards to the schedule, we were not bound by an occupancy deadline nor were we constrained by financing or banker's requirements. However, we, the Amoco team, had what I thought was an even stronger incentive. The chairman was vitally interested that this project be executed as quickly and inexpensively as possible and with no accidents.

Schal, with our input, laid out an aggressive but doable schedule. We factored into the original plan sub-contractors' proposals, planned material delivery dates, an estimate of the impact of weather, and realistic start dates.

Rick will talk finally about the details of the schedule.

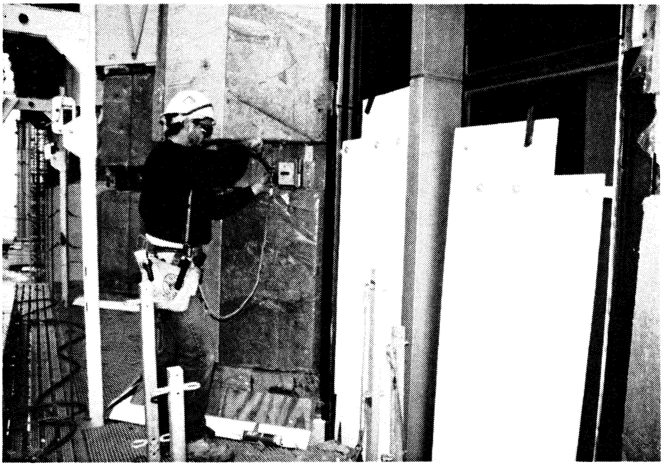

Fig. 17-1 - View of worker using tethered tools and of temporary plywood window protector.

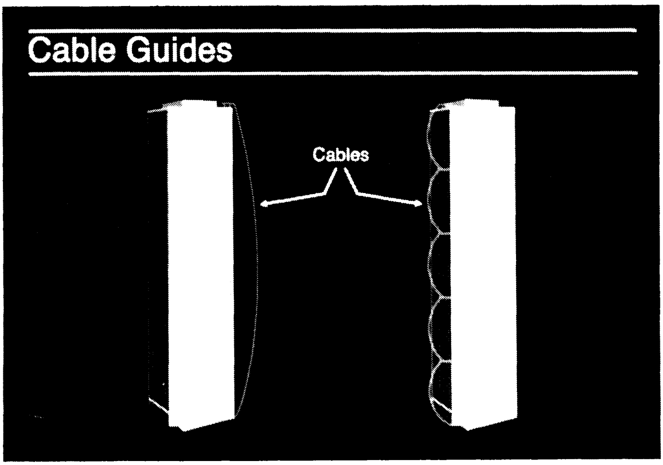

Fig. 17-2 - Extent of cable guides used to restrain cables.

CHAPTER 18
CONSTRUCTION SCHEDULE
BY
Rick Cantalupo
General Superintendent, Schal Bovis, Inc.

I guess you are wondering how long this took.

Canopy design was started in July 1988 and completed in March 1989. That had a duration of nine months. The actual erection of the canopy was started in March, 1989, and went through August, 1989, which took six months.

Design, fabrication, and delivery of the stages was started in July of 1989 and completed in March 1990, with a duration of nine months.

Erecting the stages and the monorail was started in March, 1990, finished in May, 1990, and took three months.

We began construction of the conveyor in June 1990, and finished in July, 1990. It only took one month.

The grillage installation was started in January 1990, and was completed in June 1990, which was six months.

Stone quarrying, fabrication, and the delivery of the stone was started in August 1989, like many other activities and it took until July 1991, which was a 24-month duration.

The design, fabrication, and delivery of the hoist was started in July 1989 as well and finished in July 1990, having a one-year duration. To erect the hoists, they were started in February 1990, and completed in March 1991, and that was 13 months.

Prior to the hoist running, we did a mockup at the north wall and that mockup took two months. It was started in May, 1990. Once the hoists were in operation, we began to install stone from the third through the fifth floors. It was started in August, 1990, and finished in September, 1990, taking one month.

Stone installation from floors 6 through 79 was started in September 1990, and we topped out in September of 1991, taking 12 months. We essentially erected at an average rate of a floor-and-a-half per week. Some weeks were better than that; some were a little less.

Stone installation at the top of the building, because there was also an aluminum cornice involved, started in September 1991, and was finished in November, 1991, which took two months.

The base installation of the stone which also involved some of the plaza rework was started in November, 1991, and finished in May, 1992. It took six months.

The caulking started when the stone was completed in November, 1991, and was finished in June, 1992.

To remove the hoist, we began in November of 1991 and it ran through July of 1992. It was an eight-month duration.

Removing the grillage took six months and then to dismantle the canopy took five months.

Now Ian and Roger will talk about post-construction conditions.

CHAPTER 19
WHATEVER HAPPENED TO THE MARBLE
BY
Roger Hage
Vice President, AmProp Finance, Inc.

The project team did live up to our expectations and we were able to successfully operate the building and reclad it at the same time. We had no major complaints from any of the tenants. We had some minor complaints like cables slapping at the windows, and those complaints were handled immediately. But no major complaints at all which is really, I think, outstanding. I think the team did an outstanding job.

One of the major questions that I have always been asked is "Whatever happened to the marble?" As you can see, we made one heck of a lot of bookends, examples of which are shown in Fig. 19-1.

Seriously, we did a number of things. One, we donated marble to Lashcon, Inc. and the main purpose of that was to teach skills to disabled individuals on how to do fabrication of small stone items. If you have not, please take a look at the exhibit that we have outside. All of those are for purchase. It is from an independent business. A small quantity of marble went to that.

Fig. 19-2 basically shows what happened to the rest of the marble. We ground up the marble. We donated some of the marble to a local university for their use in a water feature pond. The rest of it was ground up into small pieces and taken to some of our refineries, notably Whiting Refinery, and used for landscaping some of the grounds in the refinery area.

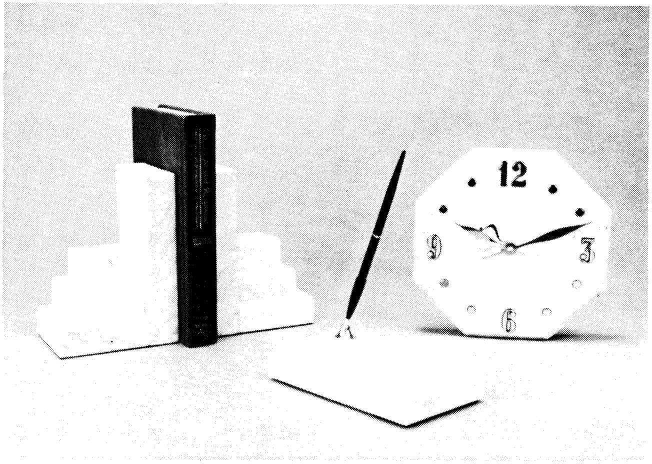

Fig. 19-1 - Examples of the use of some of the marble panels removed from the building.

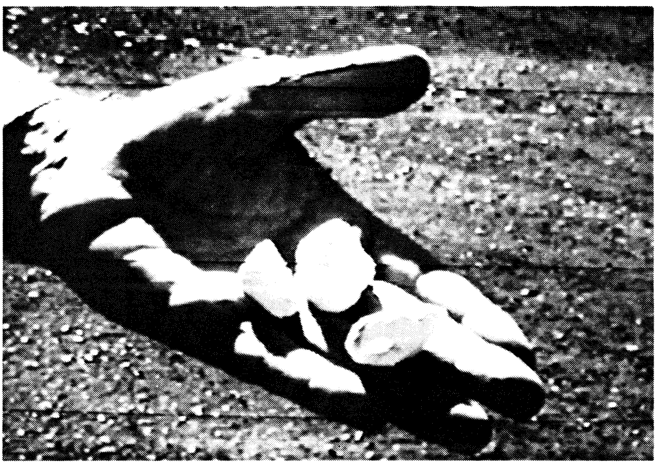

Fig. 19-2 - The vast majority of the marble panels were ground up into pebbles and used as landscaping material.

CHAPTER 20
LESSONS LEARNED FROM THE RECLADDING OF THE AMOCO BUILDING

BY
Ian R. Chin
Vice President and Principal
Wiss, Janney, Elstner Associates, Inc.

Several lessons were learned from the recladding of the Amoco building. The lessons learned that can help members of the building industry avoid the occurrence of a similar situation on buildings constructed in the future include the following:

1. Know the Materials Being Utilized

Marble has been successfully used as a building material for hundreds of years, and as a consequence its general physical properties and in-service behavior are known. However, the successful historical use of marble was based upon it being utilized in thick blocks or as thick panels of about 4 in. or greater in bearing type walls.

On the Amoco building, 1-1/4 in. (3 cm) thick, non-load bearing marble panels were used. This use of thin marble panels on buildings in the late 1960's was relatively new and, therefore, the in-service behavior of this use of thin marble on buildings had no history and was not completely known.

Designers, therefore, did not have much information available to assist and guide them with the design. As Roger pointed out the Amoco design team utilized the best available information, and material tests were performed. However, the available information utilized was limited, the testing performed was generally without precedence, and as a consequence failed to predict the behavior of the thin marble panels on this high rise building. Designers should, therefore, be careful with materials that have limited information predicting their in-service performance and behavior.

2. Verify that the materials supplied meets the project requirements

Tests were performed by Amoco on many different marbles during the marble selection process, and a marble with the desired aesthetic appearance and with a minimum flexural strength of 1,400 psi was selected. The structural design of the panels was based upon this minimum flexural strength. However, the flexural strength of the marble supplied and installed on the building was significantly less than the

minimum specified flexural strength and as a consequence the marble failed and had to be removed from the building.

Production testing of the actual materials being supplied should, therefore, be performed to verify that the materials being supplied meets the requirements of the design. This testing should include testing of samples removed from stone blocks at the quarry prior to fabrication of the panels from the blocks and of randomly selected panels cut during fabrication.

3. Utilize Team Effort During Design

The input of the aesthetic designer, the structural designer, the contractor, and the material supplier should be obtained and utilized to develop the design. This approach will help to minimize conflicts and to minimize the development of problems during fabrication, construction, and in service behavior.

4. Connections should be properly designed

Connections should be properly designed to withstand the design loads with an adequate safety factor. The connections should also be mocked up and tested to verify conformance with the design.

5. Contributions to the Building Industry

The work performed during the investigation of the original marble panels, and during the design and construction for the new granite panels has resulted in the following contributions to the building industry:

a. Guidelines for the design of kerfs in stone panels.

b. Procedures for preconstruction accelerated weathering testing of stone to obtain an indication of the initial and future flexural strength of the stone.

c. Procedures for testing of connections in stone panels.

CHAPTER 21
PANEL DISCUSSION

Question: You talked a little bit about some difference in capacity of kerfs in the stones. Could you expand on that a little bit for us?

Answer: (Jack Stecich) Fig. 12-8 (in Chapter 12) shows how a kerf connection behaves under outward wind load. When the stone panels are pulled outward, the shelf angle deforms and rotates outward which causes the bottom leg of the shelf angle to bear against the outside edge of the stone panel below - the bulb is just pulled away from the stone. This produces bending moment in the lower kerf that is greater than if the bearing point were at the bottom of the kerf. At the upper stone panel, the rotation causes the end of the insert to bear against the stone deep within the kerf, which reduces bending moment and results in a stronger connection. This condition, which was verified by laboratory tests, explains why the upper stone panel is stronger than the lower stone panel and why the use of a bulb at the tips of shelf angle type connections inserted in kerfs does not increase the capacity of kerf connectors for outward wind load. With inward loading, there is no rotation of the angle because the horizontal leg of the shelf angle bears against the clip angle and the bulbs at the ends of the inserts bear on the stone at the bottom of the kerf which makes the connection stronger than if the connector bears on the stone at the top of the kerf.

Question: In your investigations and review, did you come to any conclusions about how many other buildings in this area might have this kind of a problem and have any other buildings gone through a similar situation?

Answer: (Ian Chin) As previously mentioned by Roger, the Chase National Bank in Rochester, New York had similar white Carrera marble panels on its facade that had to be removed for basically the same reasons. That building was reclad with aluminum. There is a small building on the outskirts of Indianapolis, Indiana, that had a similar marble cladding on it that also had to be removed and replaced, and there is another major, high-rise bank building in Indianapolis, Indiana, where they are having some similar problems. So, there are quite a few buildings where this condition has occurred.

Question: I have a question regarding the design. But first a general question on the stone. Would this have been a problem had the marble been 4 inches thick? This is question number one. And then, is the hysteresis phenomenon isolated to the marble family of stones or should there be some testing done in regard to limestone or granites?

Answer: (Ian Chin) Would this condition have happened had the marble panels been 4 inches thick? Based upon the information we have at this time, we believe that the answer to that questions is no.

The hysteresis behavior, as far as permanent expansion and interlocking of the grain structure, may be unique to marbles. However, the accelerated weathering testing we have performed to date has revealed that the Mount Airy granite used to reclad this building may have a ten percent strength loss with time. Similar tests we have performed on other stones have also indicated a potential strength loss. Accelerated weathering tests should, therefore, be performed on all stones to obtain a measure of their potential strength loss due to weathering.

Question: Does hysteresis also require the presence of moisture to get the bowing?

Answer: (Ian Chin) Yes, during a test that we performed in the laboratory which basically, consisted of cutting a hole in a refrigerator door and placing a marble panel in the door and exposing the stone to temperature cycles, we did not get the panel to bow for a long time. But after we introduced moisture on the surface of the stone and continued to cycle it, we did get bowing.

(Jack Stecich) The exposure to temperature cycles also causes loss of strength. It takes the addition of water for bowing to occur, but in my view as a structural engineer, you should be very concerned about strength loss. This strength loss will occur even if water is not placed on the back side.

Question: With a thicker panel would there be a development of internal stresses that resist the hysteresis phenomenon.

Answer: (Ian Chin) There are thick white marble panels on buildings in Chicago and in Washington, DC, that have not bowed, which may indicate the presence of an internal mechanism in thicker panels that prevents bowing. However, that does not mean that they have not lost strength.

(Jack Stecich) Plus you have 4 in. working for you instead of 1-1/4. A 4 in. thick panel is stronger to start out.

Question: Did I understand that you said that caulking was the last operation performed during the recladding?

Answer: (John Dombrowski) The joints were left open and were not sealed until all for the granite panels were installed on the building. Since the cladding is actually a veneer and not a curtain wall, any water that got in would be minimal as far as destroying anything or causing any corrosion. So we decided that it would be most efficient if we left it until the end of the project. Also, we would get a better seal on the building, that is, everything done at the same time versus doing one joint then going up and installing stone and installing caulk. There were some limitations as far as how many people we could have on the stages and therefore bringing another crew in to caulk was very time consuming and very expensive.

Question: What about controlling condensation on the backs of the panels?

Answer: (Ian Chin) There is, if you remember the chevron-shaped steel columns behind the panels. At the tips of the chevron-shaped columns at the junction of the window, there is a vapor barrier that prevents interior moisture from getting into the cavity. So the problem of condensation on the back of the panels is not of concern.

Question: There are times in Chicago when you wake up and there will be dew on the lawn, your back yard, front yard, whatever. Condensation knows not where it comes from. If it reaches below the dew point, it will form. One would think that some provision for condensation would have been dealt with here. When there is condensation on the back of the panel, where is that going to go?

Answer: (John Dombrowski) Since there is a cavity, an air space in between the panels and the columns, there are some leaks from the structural steel and the connection between that and the window-washing track, there is some stack effect, some air pressure coming out of the building which now goes up and down the cavity. The maintenance crew previous to this project drilled holes in the window-washing track to vent out that air. Therefore, we have constant circulation of air within the cavity and any condensation that would build up would be evaporated through that air movement.

Question: Since you really had an unventilated space behind those chevrons, a steel column forms a vapor barrier on the inside of the wall and the stone that you created is truly a barrier system, not a drainage system.

Answer: (John Dombrowski) I think it is very important to realize that during the testing phase of the project, at least 50 panels were removed from the building and at none of these locations did we see any evidence of water having accumulated behind the panels. The fact that the building is 16 years old and there was no evidence of moisture behind the panels convinced us that we would not have a condensation problem behind the panels.

Question: On the connections where you have the picture where you had the top and the bottom slot, these slots are continuous throughout the panels, top and bottom, and not just half-moons cut in them. Again going back to the water problem, the vertical pictures of the cracks came in the bottom of the stone, not in the top of the stone. Is that correct? My question is: is it possible that any water accumulated in the top of the stone and froze in there and pushed that back piece out. You said it had to do with the wind sucking out on it. We have had problems with water freezing and popping out. That is the question I have.

Answer: (Ian Chin) This is in the original stone that you are talking about, right?

Question: On the original stone, yes.

Answer: (Ian Chin) The cracking at the connections that we saw in 1985, 55 percent of the cracks were in panels below the connections and 45 percent where in the panel above the connections. In the original design, the kerfs were filled with sealant so there was no cavity in the kerf for water to collect and freeze and cause that type of cracking.

Question: So you mean that the original slots were caulked continuously?

Answer: No, the original panels had 8 in. long moon-shaped kerfs that were fully caulked.

Question: Okay, but in addition to the face of the panel?

Answer: (Ian Chin) Yes.

Question: I wondered, Roger, what you look for in the life of the new cladding system? And then I have a question about how did you close up the panels where the hoists and monorails were attached and make a connection that appears invisible now?

Answer: (Roger Hage) We certainly hope that the life of the new cladding system will be as long as the building exists, the steel or anything else. We hope and I think through all the testing we are pretty certain that it is not going to need to be replaced before the building is demolished. One thing I might add on that is that Amoco will continue to monitor and evaluate the new cladding. We have employed John Logan to do some non-destructive tests through sonic devices. I think that one thing in working for Amoco over the years, they will not neglect it. That is a key to maintaining any building and its longevity.

Question: That second question was about the closure panels.

Answer: (Roger Hage) The simplest way to say it is that the owner, the designer, and the contractor worked together to figure out a way of putting in a closure panel. Basically, it was done by countersunk holes in the thick granite using through-bolts that were later concealed with absolutely properly designed plugs. This is one of those situations where the designer had an idea, or maybe the idea started with the contractor, I don't know, but it does not matter because it went back and forth, back and forth, and at the end of the day we worked out a panel that had proper design strength that had good life and that the contractors here could get onto the building. This was thought about from the very start. It was not done at the last minute. But I am pretty sure that for the most part, Bill, the closure panels were done with a countersunk hole and a bolt that was filled up with a plug.

Question: Jack, this is probably for you. Failure is being defined these days in specifications as permanent deflection, dislocation, or fracture. When you showed a picture of the test of the extruded stainless steel being pulled against a steel bulkhead, testing mainly the slip in the slot, in your experience in what you are dealing with now with ASTM, what are we trying to do? Are we trying to break the stone, or fail the clips? Are we working toward a brittle or ductile failure in these connections?

Answer: (Jack Stecich) Good question. The deflection that you saw in Fig. 12-13 occurred at a load equivalent to 750 psf. The steel shelf angle had yielded, but the yielding did not start until 5 or 6

times design load. Failure of the steel connection should be ductile. I think an appropriate design criteria is to use the normal factors of safety adopted by the steel industry. This criteria will keep the member below yield under design loads. Stone failure will be in a brittle mode so a higher factor of safety is required. You cannot prevent the stone failure from being brittle, but you can design the stone to not fail at design loads. The steel will have lots of ductility; it will go along for the ride. The designer's task is to design the brittle element strong enough to not fracture at design load. This is done with a higher factor of safety. The factor of safety should also address loss of strength of the stone, variabilities of stone strength, variabilities of tolerances, and variabilities in wind loadings.

Question: Will you still try to maintain a ductile connection?

Answer: (Jack Stecich) The connection will remain ductile because the steel element , which is the weaker, is ductile.

Question: In your testing in terms of stone strength loss, does stone continue over time to lose strength? I think my question is really, it seems evident in the marble that it was continuing to lose strength because from your initial investigation of the building and then your investigation in 1985 showed that it was continuing to lose strength. In your testing of granites, have you seen that it sort of reaches a strength loss and then sort of stabilizes or does it continue to lose strength?

Answer: (Jack Stecich) We see that the line after about 300 cycles tends to flatten out, indicating that strength loss is stabilized after 300 cycles. On the other hand, the original marble showed tendencies to continue to decrease in strength after 300 cycles of testing. As a matter of fact, in our recommendations we told Amoco that it was our belief that a very small percentage of the marble in the building may continue to deteriorate with time to the point where it may crumble in your hands.

Question: It seems very curious to me in terms of what you saw in the results of the staining on these sealants. Had the manufacturers done their standard tests for staining on the stone determined that this sealant would not stain the stone? This is very important to us as design professionals. We ask them to conduct staining tests on stones to make sure we do not have a problem and they say that they run these tests and it does not stain.

Answer: (Ian Chin) In my opinion there is no reliable laboratory test available right now to verify that your stone is not going to be stained by sealant. Does that answer your question? In other words, the manufacturers claim that they have a staining test but I do not think that test is reliable.

Question: I guess what I am asking is if you would have gone to initially a GE or a Dow in terms of silicone and you said, "We want to use a certain type of granite and we want you to verify, or perform, or tell us, that your sealants do not stain." They say, "We have some stain tests and it's okay." Now, you went into the next series of tests which are a little more conservative. You, in fact, had some time to make a decision. You put sealant on the actual stone on the actual building where it was going and six months later you saw some dramatic staining from some of the sealants. What I am trying to find out is, did initially the manufacturers feel their sealants were not going to stain that stone or did they not know?

Answer: (Ian Chin) The first series of tests that John showed you were performed at the Wiss, Janney, Elstner Associates, Inc. laboratory in Northbrook, IL, using over-the-counter materials and using a sealant contractor to fabricate the test setup, and you saw the dramatic findings in that testing. For the tests that John performed on the building, Amoco called in the sealant manufacturers, and told them to do what they thought was necessary to make their sealant look good and work in this particular building. Amoco also informed them about the laboratory tests that we had run, and my recollection is that none of the sealant manufacturers told us that their sealant was going to stain the stone. Let me emphasize that sealant manufacturers do have a lot of tests and some of them are very good, but I do not think that the staining tests that they currently have provided a reliable measure of whether or not the sealant is going to stain the stone.

Question: What is the plastic sheeting behind the panels?

Answer: (Ian Chin) The plastic sheathing was originally used to wrap the insulation behind the panels.

(Roger Hage) That plastic is a mystery. We surmised that it was there simply to hold the insulation in place during the installation of the original marble panels. The replacement pieces that we put

on during the installation of the new granite panels were placed once again to hold it in place during the installation. That is really the essence of why it was there.

Question: Was the insulation bought wrapped?

Answer: (Roger Hage) No. We bought the insulation and custom covered it.

Question: Various experts have different safety factors for stone. There are people that go up to a factor of safety of 8. Now, listening to this lecture, it seems that you are accepting a factor of safety of 4. And I guess that is who you talk to more than anything else. Is there any reliable factor of safety that somebody can depend on. Now, Ian, I am sure that you have read the various literature that has been written on this thing. Do you have any relative guess about what a reliable standard is?

Answer: (Ian Chin) It depends on the stone. At the present time, we utilize the safety factors that are recommended by the various companies that fabricate stone or mine stone. For instance, the Indiana Limestone Institute recommends a safety factor of 8 for limestone. The safety factors that were used in the design of the granite panels on the Amoco building, as Jack pointed out, were recommended by the National Building Granite Quarries Association. The Marble Institute of America recommends a safety factor for marble of 5 for flexure and 10 for concentrated loads. So it all depends on the type of stone. I am aware that many consultants have written papers on safety factors of stone using coefficient of variations, however, based upon the knowledge that we have gained from this project, our recommendation would be to use the safety factors that are recommended by the various stone organizations for their particular stone.

Question: Based on the track record of the material itself, naturally you expect the granite to have a better factor of safety than the marbles or limestones in that they are a lot more resilient flexurally. Is that the answer? There is one more question regarding stack action in cavities of high-rise buildings, it has always been my experience that you stop off the cavity every four or five floors so that you minimize stack action in the cavity of the exterior wall, and at that point you deal with condensation drainage so that you do not build up a stack of water condensation behind the stone or any material for the whole length of the tower. Now, I assume that on this building, they had in the recladding and in the initial instance done this protection process and it was redone in the recladding procedure. Is that correct?

Answer: (Ian Chin) No, it was not done in either case.

Question: You mean the whole thing is open?

Answer: (Ian Chin) The cavity is partially open. The cavity is partially blocked off at each panel by the continuous shelf angle. We thought about it in the recladding process. We examined the original panels and as John pointed out, found that a similar condition had existed on the building for 16 years without any condensation problems. So we decided just to go with what had proven to work successfully.

(Roger Hage) On that one issue, it is interesting because there were a lot of theories what was causing the bowing of the marble. One of the theories was that the bowing occurred because stack action in the cavity was pushing against the marble and forcing it to bow. In investigating this theory we did some testing of that and measured the actual stack action in that cavity, and found that the stack action forces was very low and did not cause the bowing.

Question: In exploring the issue of sealants a little bit further, one of the biggest arguments for silicones obviously is their long-term durability. One of the arguments against in relation to stone is the staining issue. One of the arguments against the polyurethanes is, I am assuming you are talking about a two-component material here, is the quality control issue during installation and, more recently, we have seen a lot of problems with the reversion in polyurethanes. I want to ask how you have addressed those issues and how soon do you anticipate recaulking the building?

Answer: (Roger Hage) That is what brought on all the testing because we as operators of the building, of course, wanted silicone. The designers of the building wanted polyurethane, and this was not an easy issue as you may well understand because of the quantity. We were inundated. The operating people were inundated by the manufacturers, saying that the designers don't know what they are doing and that you really need silicone and so forth. It is fortunate that we were one entity working together to achieve the final goal and that is what brought on the second set of testing. We were hearing this controversy. As an operator, I don't want to caulk this building again. The silicone people were telling me I don't have to caulk it for ten years, the other sealant people were telling me the same, and so the

second set of testing was performed to check the sealant manufacturer's claims. Certainly, we did not want to have all of our stone with window frames around it from an aesthetic standpoint.

(John Dombrowski) We also insured ourselves that the caulking that we received for the building was all from the same manufacturer's batch and so he ran a special batch just for us. We tested that batch at their facility. We tested every mix that the contractor made on the jobsite. He put a sample in a plastic cup and he sent it to the manufacturer. He tested those. We secured a ten-year guarantee for both the installation and labor and for the material durability.

(Bernie Simmons) Each mix was definitely located on the building so we know that if there was a failure, whether it is due to cohesion or adhesion loss, we knew exactly where that caulk was installed and that was because of the quality control program.

(John Dombrowski) We also came back after the installation and did pull tests at random sites on the building to insure ourselves that we had adhesion and cohesion.

(Ian Chin) I have looked at many buildings up and down Lake Shore Drive and I do not see any significant difference in longevity, at least in that locality between polyurethanes and silicone sealant.

Question: Was this a two-part again?

Answer: (Ian Chin) Yes, it was a two-part.

CHAPTER 22
CLOSURE

Ian R. Chin
Principal and Vice President
Wiss, Janney, Elstner Associates, Inc.

On behalf of the Chicago Committee on High Rise Buildings, I wish to thank Amoco for granting the Committee permission to present this seminar and for their active and essential participation in the seminar. I also wish to thank everyone in attendance for taking time off your busy schedule to spend the afternoon with us and to listen to our presentation.

The goal of this seminar is to provide information on why the original marble on the Amoco building failed and on how the new granite panels were designed and constructed so that designers of future buildings can use the information to avoid similar failures. The information was also provided to assist owners of existing buildings to recognize if they have similar conditions on their buildings and how to address such conditions if they do. We have provided the information, please use it to your advantage.

We thank you all again for coming to the seminar, and hope that you have a safe trip home.